ECONOMICS OF BRIDGEWORK

A SEQUEL TO BRIDGE ENGINEERING

BY

J. A. L. WADDELL

C.E. (Rens. Poly. Inst.); B.A.Sc., Ma.E., D.Sc. (McGill Univ.); D.E. (Univ. of Neb.); LL.D. (Univ. of Mo.); Kogakuhukushi (Doctor of Eng., Imp. Univ. of Japan)

Correspondant de l'Institut de France dans l'Académie des Sciences; Corresponsal de la Sociedad de Ingenieros del Perú, Knight Commander of the Japanese Order of the Rising Sun; Membre de la Société de Bienfaisance de la Grande Duchesse Olga de Russie, Consulting Engineer, New York City, Member of the American Society of Civil Engineers; of the American Institute of Consulting Engineers; of the Franklin Institute; of the Institution of Civil Engineers, London; of La Société des Ingénieurs Civils, Paris; of the Engineering Institute of Canada, of the Western Society of Engineers, of the Rensselaer Society of Engineers, of the Society for the Promotion of Engineering Education; of the American Association for the Advancement of Science; of the American Society for Testing Materials, of the International Society for Testing Materials; of the American Railway Engineering Association, of La Société de Géographie de France, of the Phi Beta Kappa Society; of the Tau Beta Pi Society, of the Sigma Xi Society; of the National Conservation Association; of The National Economic League, and Honorary Member of the Engineers' Club of Kansas City; and of the Société Internationale d'Études de Correspondance et d'Échanges "Concordia," Paris

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TO THE

Académie des Sciences

OF

L'Institut de France

THE UNIVERSALLY ACKNOWLEDGED

LEADER OF THE WORLD

IN ALL LINES OF SCIENTIFIC THOUGHT

AS A MARK OF

THE AUTHOR'S GRATEFUL APPRECIATION OF THE

PROUD DISTINCTION CONFERRED UPON HIM

BY FRANCE

THROUGH ADMISSION TO ITS RANKS

ON DECEMBER 16, 1918,

AND AS AN EVIDENCE OF HIS PROFOUND ADMIRATION

AND REGARD FOR THE ENTIRE FRENCH NATION

THIS TREATISE

HIS FINAL CONTRIBUTION IN BOOK FORM

TO ENGINEERING LITERATURE

WITH THE ACADEMY'S KIND PERMISSION

IS MOST RESPECTFULLY

DEDICATED

PREFACE

THE author's object in foisting still another bridge book upon his long-suffering brother engineers is twofold, viz.:

First. A desire to leave behind him for the benefit of the next generation of bridge specialists, in shape readily available for use, a solution of all the major economic problems in bridgework and an extensive treatment of most of the minor ones. He has endeavored to cover every possible economic question of importance; and, if any such be omitted, it is because he did not recognize their existence. To what extent he has succeeded in this endeavor can be determined only by the reception which the book meets from the engineering profession.

Second. To show to specialists in all branches of the engineering profession the type of book on economics which, in his opinion, each specialty needs and how broad and thorough, if possible, should be the treatment of the subject.

The author feels that he is the logical person to write a book upon the economics of bridgework, because he has been investigating the matter for thirty-seven years, and has dealt at length, upon more than twenty different occasions, in books, pamphlets, and memoirs, with various economic topics on the subject of bridge designing, as can be seen by referring to the list of the said writings given in the Appendix.

An explanation but not an apology is needed for the fact that certain portions of "Bridge Engineering" have been copied in a few of the chapters. As that work from start to finish is impregnated with the fundamental idea of economy, and as this book is intended to cover the entire field of bridge economics, it was necessary to include herein the gist of all that appears on the matter in that work, as well as the substance of all the author's economic writings that are subsequent to its publication. Wherever anything previously written on the subject could advantageously be modified or enlarged, this has been done; but where it could not be improved upon or augmented, the only thing to do was to copy it *verbatim*. Hence, should any reader see any portion of "Economics of Bridgework," which appears familiar to him, if he has read this preface he will understand the reason therefor and will, the author trusts, excuse the occurrence.

In proof of the previous statement that the treatment throughout "Bridge Engineering" is based on the conception of economics, there is offered the following extract from a review of that work written by Albert Reichmann, Mem. Am. Soc. C.E., and published in the October, 1916, issue of the Journal of the Western Society of Engineers.

"In a larger sense, this work is a treatise on the subject of 'Economics of Bridge Engineering.' This train of thought can be traced throughout the entire work, all subjects being treated fully and in a broad way, and with the idea of value constantly brought out."

There is a salient feature of the present book to which attention should be called, and that is the scarcity of treatment of economic problems by pure mathematics, nearly all the formulæ given being semi-rational, semi-empirical. The reason for this is that the conditions affecting most of the economic questions in bridge designing are far too complicated to lend themselves to solution by mathematical manipulations. This fact has not always been recognized by engineering writers; for, during the last three or four decades, numerous attempts have been made to settle important economic bridge-problems mathematically, with the result that the conclusions thus drawn have proved to be erroneous and generally totally useless.

It is prognosticated that, in the future, there will be no such fundamental changes in methods of bridge design and construction as to render really incorrect the numerous economic conclusions reached herein; because bridge building has now become fairly well systematized. About the only important change in sight is the adoption of high-alloy steel for long-span superstructures; and the effects of this have been duly anticipated. The reasons for this belief are that the investigations upon which the book is based were in general predicated upon fundamental engineering principles that do not vary; and that the effects of possible changes in unit prices of materials in place and in other general conditions affecting design have been indicated. While it is true that an abnormal variation of a temporary nature in the unit price of some important material of bridge construction, some peculiar feature of structure-location affecting erection, or possibly some other cause, may change somewhat the findings stated herein, it must be remembered that, as pointed out in several places, up to a certain limit a considerable divergence from the exact economic condition will usually cause no serious augmentation of total cost. This is the saving clause which renders the author's economic conclusions sufficiently dependable for the future as well as for the present.

The story of the writing of this treatise is as follows:

In the summer of 1916, when "Bridge Engineering" was issued, the author recognized that there was one very important bridge subject that he had not completely covered, viz., economics; and he thereupon listed ten major economic problems at that time unsolved, and came to the determination that they should no longer be left in that condition, if he could prevent it. After repeatedly failing to interest any other investigator in

the matter, he himself undertook the task and accomplished it, at least to his own satisfaction, in four years of hard work.

The following is a list of the said economic questions arranged in the order of their final solution:

1. Economics of Steel Arch-Bridges.
2. Comparative Economics of Cantilever and Suspension Bridges.
3. Economic Span-Lengths for Simple-Truss Bridges on Various Types of Foundation.
4. Possibilities and Economics of the *Transbordeur.*
5. Comparative Economics of Continuous and Non-Continuous Trusses.
6. Comparative Economics of Wire Cables and High-Alloy-Steel Eye-bar-Cables for Long-Span Suspension-Bridges.
7. Economics of Reinforced-Concrete, Steam-Railway Bridges.
8. *De l'Emploi Économique des Alliages d'Acier dans la Construction des Ponts.*
9. Bridge *Versus* Tunnel for the Proposed Hudson River Crossing at New York City.
10. Economics of Movable Spans.

As fast as these economic investigations were finished, they were separately incorporated in chapters of this book, occasionally *in toto* but generally with slight modifications or omissions.

In the preparation of a few portions of the manuscript, the author has received valuable assistance from the following engineers, to whom he herewith tenders his hearty thanks: Messrs. Thomas E. Brown, Leon L. Clarke, Watson Vredenburgh, Thomas Earle, C. M. Canady, Harry K. Seltzer, Joseph J. Yates, Frank W. Skinner, Charles F. Loweth, Carl S. Heritage, J. G. Chalfant, Vernon R. Covell, Edward A. Byrne, J. B. W. Gardiner, L. H. Beach, P. S. Bond, A. H. Sabin, and Shortridge Hardesty.

The special portions of the book whereon these gentlemen have aided are indicated throughout the text, excepting in the case of the author's assistant engineer, Mr. Hardesty, who helped with certain of the calculations and gave the entire treatise a general check.

It may be of interest to some readers to learn how Chapter XLIV on "Economics of Military Bridging" happened to be included in the book; for it is a subject concerning which, before the said chapter was written, the author knew practically nothing. Incidentally it came to the attention of Major R. W. Lewis—a young engineer who served with distinction in the American Army on the battle fields of France, and a son of Col. I. N. Lewis, the famous inventor of the machine gun which bears his name, and which so greatly aided the Allies in winning the war—that this book was in course of preparation. He thereupon called on the author and asked if he would consider favorably a suggestion to insert in the treatise a chapter prepared officially by the Engineer Corps of the Army on the subject of

economics of military bridgework. Upon receiving an acquiescent reply, Major Lewis proceeded to the Capital, consulted Major General Beach, the Chief of Engineers, and made a like suggestion to him. The result was that the General agreed to write a "Foreword" for the chapter, and detailed Col. P. S. Bond, in consultation with some of his brother officers, to do the writing. The author deems the outcome an exceedingly valuable addition to his book; and he hopes that many of his readers, in consequence of its perusal, will be induced to take such an interest in military bridge engineering as to ensure that they shall be better prepared to serve our country in case of war than were most of the civilian engineers when the call to arms came in 1917.

It is intended that this shall be the last technical book which the author will ever write, for reasons explained in the concluding chapter; and he hopes that it and its immediate predecessor, "Bridge Engineering," will, for many a year to come, prove of real service to the engineers of his specialty, in the advancement of which, almost ever since graduation forty-five years ago, he has taken an intense and absorbing interest.

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CHAPTER I

INTRODUCTION

UPON the scientific application of the principles of true economy in all lines of activity will depend the success of every one of the great nations in the world-struggle for business-supremacy which is about to follow the final close of the Great War. This statement is peculiarly applicable to the United States of America, which is always unavoidably handicapped by the high cost of labor, and at present is unnecessarily and stupidly hampered by an epidemic of strikes and a wide-spread insane desire to shorten working hours far below the minimum limit requisite for adequate production of the necessities of life for our own country alone.

Until mankind recognizes that labor is a blessing—not a curse—progress will be slow, and the development of the world in all lines will be seriously impeded. No healthy man or woman, or even a child of school age, is injured by eight hours per day of fairly-strenuous physical or mental exercise, provided that its character be suited to the individual's age, capacity, and taste. When one is healthily tired upon the expiration of his daily task, he is in condition, after a very short rest, to enjoy his food and recreation; but any person having no settled occupation nor any regular duties to perform is an unhappy, discontented individual who not only spoils his own life but also interferes with the enjoyment and well-being of all persons with whom he comes in contact.

A real love for work *per se* should be inculcated in every child by its parents and teachers; and extra work should never be given as a punishment, because so doing would engender a distaste for labor. All work should be made as pleasant and interesting as possible, not only for children but also for adults; and if anyone cannot learn to like the occupation to which he has been assigned, the character of his employment should be changed from time to time until he finds a niche into which he fits comfortably. This development of a love for work in the young is specially important in relation to study; because, when an individual once becomes truly interested in his occupation, be it either mental or physical, his life's battle is already more than half-won.

For several years the U. S. A. has had a unique opportunity to become in all lines the leading nation of the world; but alas! it has not recognized the existence of this important privilege or taken the steps necessary for its utilization. Latin-America is knocking at our door asking us to do business; the nations of Asia, Africa, and Australasia are eager to enter into commercial relations with us; and the peoples of war-afflicted Europe need both our manufactures and our raw materials. But what foreign business can we do when Americans in general are willing to work only six hours per day and five days per week—and even then with lessened efficiency? That amount of effort is insufficient to provide for home necessities; and, therefore, it is entirely inadequate for the development of foreign trade. The beaten Huns and their deluded allies will have to work long hours for many a year to come, in order to pay the principal and interest of the immense debts which are a just punishment for their iniquitous endeavor to conquer the rest of the world and impose upon it their vicious system of "kultur"; the innocent nations whom they have despoiled will have to labor just as strenuously, in order to repair their damages and repay the money they were forced to borrow in their dire struggle to maintain national existence; and even the people of Great Britain and her colonies will be compelled to work overtime to reduce their enormous indebtedness. While all these peoples are laboring year in and year out early and late for six and even seven days per week, what will eventually happen to this country if its inhabitants cut down the hours of labor and insist notwithstanding upon maintaining an extravagant style of living? There is but one answer to this question, and that is DIRE DISASTER!

Even should the American populace awaken to the fact that it is necessary for them to work full time, the difficulty will still not be overcome; because the average personal efficiency of Americans in all lines is decidedly lower than it was in *ante bellum* days. This inexcusable evil will certainly have to be corrected; but even then our country will be handicapped, for it must not be forgotten that, notwithstanding our being the one great creditor nation of the world, the war has run us deeply into debt, and that it will require hard work and plenty of it to pay the interest and liquidate the principal.

What then can be done to increase our national efficiency to the extent which is necessary for the maintenance of the improved general style of living of the so-called working classes and at the same time outstrip our competitors for the business supremacy of the world? The answer to this momentous question is not difficult, although the accomplishment of the desideratum may prove to be far from easy. It is to develop throughout the entire country to the utmost limit both the theory and the practice of true economy in every line of endeavor. By the expression "true economy" is not meant depriving people of either the necessities or even the luxuries of life (although, truth to tell, a curtailment of the latter would greatly aid in expediting the desired result), but a universal increase in

personal efficiency—an elimination of useless effort—a systematic, scientific study of how best to accomplish all important desiderata—and keeping usefully and efficiently occupied all members of society.

As almost all the material progress of mankind is either directly or indirectly due to the work of the engineer, the importance of increasing his efficiency is paramount; and, hence, the study and development of the science of engineering economics is of prime importance. Notwithstanding this incontrovertible truth, it is a fact that in times past our universities and technical schools have almost entirely ignored this fundamental and vitally-important requirement of the engineering profession. Only a few of them provided in the curriculum any instruction at all in technical economics; and even these did not attach to the course anything like the importance which is its due.

A few years ago, the Society for the Promotion of Engineering Education, awakening to a realization of its shortcoming in this vital matter, appointed a committee of four, of which the author was chairman, to investigate and report upon the subject of "The Study of Economics in Technical Schools." After working upon the question for two years, the committee reported unanimously in favor of the subject being taught in all technical courses, and indicated the ground that ought to be covered and the minimum amount of time that should be allotted to it in the curriculum. This was as far as the other three members of the committee at that time were willing to go; but the author went farther by submitting a supplementary report advocating the preparation of an elaborate treatise on "The Economics of Engineering" by a large number of carefully-chosen specialists under the auspices of the Society. As, in the author's opinion, this suggestion of his is of national importance, he herewith reproduces *verbatim* the said supplementary report.

"This is by no means a minority report, for the writer thereof agrees heartily with all the conclusions of the committee, the members of which are now in entire accord on all matters considered, after having discussed at great length many mooted points; but it is a *supplementary* report, or something in the nature of a corollary. It is true that the fundamental suggestion it contains was made to the committee, but it was not deemed advisable to include it in the formal report; hence the writer makes it upon his own responsibility.

"Briefly stated, it is that the Society undertake the compilation into book form of a large number of monographs to be prepared by the most eminent American or Canadian specialists, one in each division or subdivision of engineering, all of the said monographs being edited and introduced by a special committee of the Society, and treating, in as complete a manner as practicable, of the economics of design and construction in the various lines. The book (which might very properly be called 'The Economics of Engineering') should begin with a full 'Introduction' explaining the *raison d'être* of the work, giving a history of its compilation, and offering suggestions as to how best it may be utilized, both in its own form and in smaller derived books.

"This chapter should be followed by one which treats fully of the fundamental economic problem underlying every important engineering enterprise, viz., its financial probabilities and possibilities—in other words, whether the project under consideration would prove to be a profitable investment.

"Following this should come, preferably in their alphabetical order, the before-mentioned treatments of the economics of design and construction in the various engineering specialties.

"Such a treatise would constitute an encyclopædia of practical information on one of the most important features of modern technics. It would be invaluable to both the practicing engineers and the teachers of engineering; also to the better class of engineering students, or those of them who are not afraid of hard work—especially post-graduate students. But in order to make its contents of use to the average student, it would be necessary to prepare from it text-books of a simple character. Should the Society favor having such a treatise issued under its auspices, a publisher of established reputation could undoubtedly be found to finance the undertaking and issue the work at his own expense—preferably without paying any royalty, in order to keep the selling price down to a minimum.

"A short time ago, by the request of the Dean of the Engineering Department of the University of Kansas, the writer prepared and delivered a course on 'Engineering Economics' in three lectures to the engineering faculty and students of that institution. The University has lately published these lectures in pamphlet form; and the writer has placed a few copies thereof at the disposal of this meeting. The first lecture deals with the fundamental economic problem of finance before-mentioned; the second treats in general detail of the economics of bridge design and construction; and the third is a compilation of certain data furnished by the courtesy of a number of prominent specialists, outlining in a general way the economics of design and construction in their specialties. The writer would be willing to rewrite and expand the first lecture so as to make it serve for the second chapter of the proposed treatise; and the second lecture could be sent to each of the chosen specialists in order to indicate the extent of the desired thoroughness of treatment of his subject, and to serve otherwise as a guide to him in the preparation of the manuscript. If it were expanded by the writer so as to include the 'Economics of Steel Arch Bridges,' it might serve for the chapter devoted to the specialty of bridge engineering.

"The third lecture, which nowhere aims at completeness, would prove useful in suggesting some of the salient topics which should be covered in the writings of a few of the specialists.

"The preceding proposal is made with all due deference to the distinguished technical educators and practicing engineers of which this Society is so largely composed, in the hope that it will meet with a favorable reception and will prove to be the means of materially augmenting the amount of available technical knowledge in a line of thought which has not received, up to the present time, the consideration which is its due.

"Respectfully submitted,
"J. A. L. WADDELL,
"*Chairman.*"

This report was read by the author at an annual meeting of the Society; and, in response to his suggestion, a committee of three members was appointed to consider and report upon the proposal. At the next annual meeting that committee recommended against the Society's fathering the suggested enterprise.

Failing in this endeavor, the author, over a year ago, made to the National Economic League, of which he is a member, the suggestion that it undertake the work which the S.P.E.E. had refused; but the first-mentioned organization has not yet taken any action. Assuming that the long delay indicates that this society also intends to "pass the buck,"

the author has given up the attempt to induce his brother specialists to collaborate with him in the production of the proposed treatise on "The Economics of Engineering." Instead he presents to the profession as a possible starter for such a joint work this treatise on bridge economics, in the hope that some day American engineers will awaken to the importance of having available either a number of small books on the economics of the various specialties or else a joint work similar to the one which he has advocated, and which appears at present to be too much in advance of the times.

CHAPTER II

GENERAL ECONOMIC PRINCIPLES

THE subject of economics in engineering may properly be termed a new one, in spite of the facts that for several decades the leading specialists, in a more or less desultory way, have given the matter some attention in their designing, and that there have been several small treatises written upon the mathematical theory of engineering economics, notably Prof. J. C. L. Fish's excellent little book which bears that title, and which every progressive engineer ought to read. Besides bridgework, the only engineering specialty that has received any attention worth mentioning in respect to the important question of economics is railroading—and that field still needs a vast amount of investigation. It is a branch of engineering which, like bridgework, readily lends itself to economic study; and it is to be hoped that ere many years the economics of railroading in all its branches will be thoroughly solved by a number of America's most able railroad men, and that they will give to the engineering profession the benefit of their experience and investigations.

As far as relates to engineering, the term "Economics" has been thus defined by the author in the "Glossary of Terms" given in "Bridge Engineering"—"The science of obtaining a desired result with the ultimate minimum expenditure of effort, money, or material." It is upon the basis of that definition that this treatise is predicated.

When determining, from the standpoint of economy, which one is the best of a number of proposed constructions or machines, there should be computed for each case the four following quantities, and their sums:

A. The annual expense for operation.
B. The average annual cost of repairs.
C. The average annual cost of renewals.
D. The annual interest on the money invested.

That one for which the sum of these items is least is the most economic of all the proposed constructions or machines; but this statement is truly correct only when the costs of operation, repairs, and renewals are averaged over a long term of years; or else for a comparatively short period of time, when the conditions in respect to wear and deterioration at the end of that period are practically the same for all cases.

The principal economic investigation which occurs in engineering practice is that of determining the financial excellence of a proposed enter-

prise. It consists in showing by proper calculations its first cost, the probable total annual expense of maintenance, repairs, operation, and interest, the advisable allowance for deterioration or ultimate replacement, the probable gross income, and the resulting net income that can be used in paying dividends on the stock or other profits to the promoters. Whether any proposed enterprise, after being thus figured, will prove profitable will depend greatly on the state of the money market, the size of the project, the probabilities of future changes in governing conditions, and the personal equation of the investor. Generally speaking, if the computed *net* annual profits on the total cost of the investment (over and above all expenses of every kind, including maintenance, repairs, operation, sinking fund, and interest on all borrowed capital) do not exceed five (5) per cent of the said total cost, the project is not attractive; if it be as high as ten (10) per cent, the enterprise is deemed ordinarily good; and if it be fifteen (15) per cent or more, the scheme is termed "gilt-edged." Small projects necessitate greater probable percentages of net earnings than do large ones; and any possibility of a future reduction of income will call for a high estimate of net earning capacity. Finally, the measure of individual greed on the part of the investor will be found to be an important factor in the determination of the attractiveness of any suggested enterprise.

Such investigations as the economics of an important project should generally be entrusted only to engineers experienced in the line of activity to which the said project properly belongs; for if they be left to inexperienced investigators, it is more than likely that mistakes will be made and money lost in consequence. The professional men who generally do such work are the independent consulting engineers; certain specialists retained on salary solely for this purpose by important organizations, such as railroad companies; and engineers who are regularly in the employ of large banking houses. The work involved is of such importance that it usually commands large compensation—as, indeed, it should; because to do it effectively demands not only long experience but also good judgment and a vast amount of mental labor, both in order to make oneself capable in general and so as to consider thoroughly all the points embraced by the special problem in hand.

How short sighted most promoters of important projects can be! They imagine they can obtain expert opinion of real value without paying for it; consequently they collect a mass of scattered and divergent information, which, in most cases, is of no earthly use. Any project of importance is, of necessity, a great economic problem, and ought to be solved at the very outset by special engineering talent of the highest order.

A glaring example of the utter folly of a community in proceeding with important engineering construction without first having a thorough economic study of the problem made by a competent specialist is given in *Engineering News* of November 30, 1916. It relates to the municipal water-power enterprise on which the city of Montreal has been busy for some

years. Feeling that the work was being sadly mismanaged, certain prominent Canadian engineers "butted in," investigated, and reported upon the incomplete project. They showed that the $12,000,000 enterprise, which the city had about half finished, will fall so far short of returning a profit on its cost, and that it has so many serious defects, that it will be far better for the community to lose all it has thus far expended than to incur the additional outlay necessary to complete the work. A thorough investigation to determine beforehand whether the scheme would be profitable would certainly have indicated the futility of constructing on the lines adopted.

This fundamental economic problem of whether the proposed construction will prove to be a paying enterprise is often one of extreme complication, involving, perhaps, a determination of the character of the proposed improvement, a choice of sites or routes, a selection of uses, a consideration of æsthetics, an opinion on type or style of construction, a question of ultimate durability, a study of greatest possible convenience, a prevision of serious opposition, a prognostication of future conditions, an anticipation of prospective structural modifications, and a safe estimate of cost. The best way to illustrate such complication is by presenting a few examples of actual cases, either pending or already solved.*

CASE I

There is in contemplation a project for building a long, high, and exceedingly expensive bridge across San Francisco Harbor so as to connect the city of San Francisco with the cities of Oakland, Berkeley, and its other suburbs. This project has been a dream for at least a decade; but it is not an idle dream, because some day in some manner or other it is certain to be realized. Some ten years ago the author prepared a report for a banker on the feasibility of the project, the necessity for the structure, the possible revenue from its use, and its approximate cost; and since then several other engineers have made independent studies of the problem.

* These examples are copied *verbatim* from Lecture No. 1 of the author's series of lectures on "Engineering Economics" delivered early in 1917 to the Engineering Alumni Association of the University of Kansas. Three only of the five illustrative cases, viz., those pertaining to bridges, have been reproduced. In respect to Case 2, it is of interest to note that the suggested economic study has since been made by a Board of Advisory Engineers consisting of Col. Bion J. Arnold as terminal expert, Mr. J. Vipond Davies as tunnel expert, and the author as bridge expert. Their joint report was finished and presented early in 1919; but its contents have not yet been made public. All of the items mentioned were given full consideration; and the difficult question of how to tunnel through soft material at an unprecedented depth was solved in a masterly manner by Mr. Davies. It is hoped by the writers of the report, which consists of some 600 type-written pages and a large volume of folded blueprints, that it will be published ere long by the city of New Orleans; because it would provide the engineering profession with a good example of how to prepare a complicated economic study of a complex subject upon an unusually large scale, and how to solve several new and difficult engineering problems.

The communities interested, however, have taken as yet no sensible step towards making a thorough study of the question.

Practically none of the governing conditions are satisfactorily known; and, judging by present indications, it is likely to be a long time before they will be, unless, perchance, the leading citizens of the various communities concerned bestir themselves and prevail on their ruling bodies to join forces, raise the requisite funds, choose an engineer of national reputation (or, preferably, a board of three such engineers), arrange to allow him or them adequate compensation for expert services and all the money necessary for borings and other investigations, accord ample time for the entire work, and thus obtain a report that will settle finally all the important economic and technical points involved in the proposition.

The main features to determine are as follows, the listing being done according to relative importance:

First. The probable gross incomes from all practicable combinations of the various sources, year by year for a long term of years, and the proportion thereof that is likely to prove net in each combination.

Second. Based upon the result of this investigation, the determination of the extreme superior limit of cost for the structure for each combination of the different kinds of traffic.

Third. In view of the great depths of water in the harbor, what one or more of the various proposed or possible sites might be utilized for building a bridge within the several ascertained limits of cost.

Fourth. Of what kind of traffic it is advisable that the proposed bridge should take care.

Fifth. The character of the foundation soil as determined beyond all doubt by making proper borings, and the settlement of best depths for all pier foundations.

Sixth. The various requirements of the U. S. Government in respect to minimum span-lengths, vertical clearances, and temporary obstruction of waterway for each proposed crossing.

Seventh. The minimum clear-headway which, for a combination of all reasons, it is expedient to adopt for each proposed crossing; and the choice between a bridge with and a bridge without an opening span or opening spans.

Eighth. A safe estimate of total cost of structure for each layout that proves to be feasible, and the corresponding estimates of cost of operation, maintenance, repairs, etc.

Ninth. The time required for completion of construction of the structure for each feasible layout.

After all these points have been settled and embodied in a report, it will be easy to determine finally whether, either at the present time or within a certain number of years, it will be practicable or advisable to build the proposed structure; where it should be located; what traffic it should carry; how long it will take to build it; what it will cost for construction,

maintenance, and operation; and how the necessary funds are to be provided.

The actual conditions for the proposed San Francisco Harbor bridge, as well as they can be stated at present, are as follows:

A. There is a large possible income from passenger traffic, mainly from commutors who now use the ferry, most of which traffic would soon be diverted to the structure, provided that truly-rapid transit thereon be furnished at all times; and the said income, under present conditions, would be large enough to warrant the building of an open-decked, double-track, electric-railway bridge that would carry no other kind of traffic.

B. There is a rapidly increasing amount of automobile traffic now cared for by the ferry; and this, undoubtedly, would be augmented materially by the superior and, possibly, cheaper service of the bridge; nevertheless it is doubtful whether it would be large enough to warrant the building of separate passageways with paved floor and the necessarily-greater carrying capacity of the trusses.* It would be out of the question to let the automobiles use the same space as the electric trains; for such an arrangement would prevent the rapid transit of the latter.

C. There is an immense amount of freight crossing the water; but, for two reasons, it does not appear probable that it would ever be economical to transport it over a bridge. The first reason is the great height to which both it and its containing vehicles would have to be lifted, and the consequent expense of such lifting. The second is the greatly augmented cost of structure, due to the far larger live loads for both the floor system and the trusses, which the carrying of such freight would necessitate. It would probably be more economical either to transfer the freight by ferry, as is done at present, or to carry it by rail around the south end of the Bay.

D. There is at certain seasons a large amount of passenger traffic to and from San Francisco by certain trans-continental railroads; hence the question arises whether the passengers should be carried across in the steam-railway cars on which they travel or whether they should go over in the electric-railway cars. The objection to the latter method is the individual trouble, inconvenience, and loss of time for each passenger; while the objection to the former is the increased cost of structure due to the difference in the live loads between steam-railway cars and electric-railway cars. Of course, the former cars would have to be hauled in short trains by electric motors, so as to avoid the excessive concentrated loading from the heavy steam-locomotives.

E. The most direct route for the crossing is from Telegraph Hill to the outer end of Goat Island, and thence to near the Oakland Pier; and this is the one to which, until quite lately, most attention has been paid. Th main objections to it are as follows:

* Since this was written, the immense development of automobile travel and motor-truck traffic throughout the entire country might reverse this economic conclusion.

First. The depth of water between the city and Goat Island is excessive, thus making the pier foundations very expensive.

Second. A large proportion of the steamers using the harbor would have to pass under the structure.

Third. The War-Department requirements in respect to both horizontal and vertical clearances would be excessive for this location, because of the large number of vessels passing; and, in consequence, the cost of structure would be greatly augmented.

F. By locating further inside the Bay, the depth of water would be reduced to a reasonable amount, and the number of vessels passing the structure would be comparatively small. In fact, the farther back from the harbor-entrance the structure is located, the smaller will be the depth of water and the fewer will be the passing vessels. On the other hand, though, the greater will be the total length of structure, the farther from the center of population will be its city end, and the greater will be the distance which the passengers will have to travel.

G. Practically nothing is known about the characters of foundations that would be encountered at the various proposed locations; and no provision has been made for money to make the necessary borings.

H. It is impracticable to obtain a final decision concerning required span-lengths until a *bona fide* design, properly backed, has been presented to the War Department for approval.

I. In regard to minimum clear-headway, it is probable that the farther inside the harbor the location the less the requirement, because the smaller and less important would be the passing craft, and the fewer the number thereof. Some of them might be forced to lower topgallant masts in order to pass beneath the structure.

J. There would be a serious objection to any opening span, because of the delay which would be involved by its operation. The real *raison d'être* of the structure is rapid transit, hence to interfere with that in any way would be highly objectionable.

K. The total cost of structure would decrease to a certain point as the location is moved up the harbor, because of cheaper foundations and the consequently shorter spans; but beyond the said point it would increase because of the greater length of bridge.

L. The more expensive the structure the longer will be the time required to build it; hence it may be concluded that one of the inner-harbor locations would need much less time for completion of bridge than the Goat-Island layout. This matter of time for completion of structure possesses a double importance, because any delay increases the item of cost due to interest during construction; and by postponing the inception of operation it involves a loss of income from use.

From the preceding it is evident that the solution of the initial economic problem in connection with the proposed San Francisco Harbor bridge is one of considerable complication.

CASE II

The City of New Orleans for many years has had under consideration the building of a combined railway and highway bridge across the Mississippi River; and within the last few years the project has been seriously contemplated.

Some two decades ago the late Collis P. Huntington, President of the Southern Pacific Railway Company, and his consulting engineer, the late Dr. Elmer L. Corthell, made an investigation of the scheme of building at that place a double-track railway bridge; and they called in as advisory engineers the author and his brother, Montgomery, to estimate upon the cost of a low bridge. The death of Mr. Huntington, which occurred shortly afterwards, caused the project to be dropped; and it was never revived. The author's joint study was made with considerable thoroughness. It involved the solution of two or three problems of great magnitude that were new to the engineering profession, the principal economic one being a comparison of costs of a high bridge and a low bridge. The result was decidedly in favor of the latter.

The problem now facing the city, however, is much more complicated, involving, as it does, a combination of steam-railway, electric-railway, vehicular, and pedestrian traffics. There is a choice between two locations, one near the center of the city and the other several miles further upstream—in fact, some intermediate locations might have to be considered. There is a sentiment among certain prominent citizens favoring a tunnel rather than a bridge; and, on that account, the question of bridge *versus* tunnel will have to be considered, notwithstanding the fact that the difficulties presented by the tunnel proposition are almost insurmountable in view of the present status of engineering knowledge and experience.

The question of high-bridge *versus* low-bridge will have to be thoroughly thrashed out in order to please the public, although any truly-experienced engineer would determine very quickly in favor of the latter, irrespective of the possible opposition of the river interests and even that of the War Department.

The economic method of handling the combination of the various kinds of traffic would require some study to determine; and the best method might vary with the location of the structure.

The style and dimensions of the moving span—whether swing, bascule, or vertical lift—and the sizes of the clear opening or openings are mooted points involving a consideration of economics and other important matters. This question is complicated by the fact that the requirements ought to be dependent on the location; because at the upper one there would be very few vessels passing, while at the lower one there would be many.

The unprecedented depth for the pier foundations involves an economic study in order to ascertain the best method of sinking and founding.

The facilities for freight, passenger, and vehicular traffic afforded by

the several proposed crossings would affect the total earnings of the structure; hence this feature should receive special attention.

The grade and alignment for any proposed crossing are factors that must be included in the economic study, because they affect the cost of handling the traffic; and the matter of right-of-way may prove an important consideration.

The property damages involved by the approaches to the structure and by the shifting of existing tracks would differ materially in cost at the various possible crossings, hence this feature is one involving economics.

The choice between single-deck and double-deck entails a consideration of economics that may prove to be of some importance.

After determining the various kinds of traffic to take care of, there remains the economic problem of deciding upon the live loads for the various parts of the structure. If these are made too high, there is a waste of material involved, and the bridge enterprise will forever after be burdened with an unnecessarily large annual interest to be paid on that account; but if they are made much too low, the life of the structure will be curtailed. The saving clause, however, in respect to this adjustment is that, ordinarily, a steel bridge does not have to be removed because of overloading until the metal thereof is actually stressed at least fifty (50) per cent more than the permissible intensities of working stresses given in standard specifications for design.

This general problem of the proposed New Orleans bridge, while not so complicated as that of the proposed San Francisco-Harbor structure, is of an intricate nature, and will demand for its solution engineering ability and experience of the highest order.

Case III

There is given in "Bridge Engineering" in the chapter on "Reports" on pp. 1575 to 1581, inclusive, an economic study for the replacement of a bridge over the Mississippi River, which illustrates some of the economic questions that arise in a consulting bridge engineer's practice. In that case the point at issue was whether it would be best to build a single-track or a double-track bridge or to arrange for the conversion at some future time of a single-track structure into a double-track one. Five methods of doing the latter were suggested, and the estimates of their total first costs were made; then, at an assumed rate of interest, a table was prepared showing the total cost of each structure plus compound interest thereon for periods of five years, up to the limit of forty years. That table indicates at a glance the comparative economics of all five methods at any of the five-year periods. A diagram prepared from the said table, of course, would be preferable, as it would show more readily the comparison at any intermediate period.

In comparing the economics of several methods of accomplishing the same result, the author has advocated the method of computing and contrasting the total annual costs, including interest on first cost, upon the assumption that all the money needed for the construction had been borrowed; and this is the most logical method, although in his practice he has sometimes adopted other methods—mainly to please his clients. Some clients want to see figures of total cost instead of estimates of annual expense; and, under such a condition, it is necessary to sum up for each case all annual expenses except interest, ascertain what amount of money would produce this sum by simple interest at current rates; and add that amount to the total first cost. The case giving the least grand total would be the economic one.

An effective method of contrasting several differing types of construction for their economics is that used in Case III, viz., to assume a number of future dates, preferably those at which certain large expenditures would probably have to be made for renewals or repairs of perishable portions, and compute the grand total cost to each date for each proposed structure under the assumption that it is then put into perfect condition, and allowing standard compound interest not only on the first cost but also on all annual expenditures. A comparison of these grand total costs at the several dates adopted will indicate clearly which is the most economic of all the types of construction compared.

To those who have a penchant for using mathematical formulæ, the following economic treatment will appeal. It was evolved some forty years ago by Ashbel Welsh, past-president of the American Society of Civil Engineers.

To Find the Comparative Economy of Two Bridges of Different Cost and Durability, that will Answer the Same Purpose Equally Well While They Last

"Let C be the cost and assumed real value of one of them, T the time it will last, a the compound interest on one dollar for that time, at whatever rate money is worth to the party paying for the bridge, and L the loss on the bridge at the end of the time T, or the amount which it would take to make it as good as new. Let R be the real value of the other bridge, C' its cost, T' its duration, a' the compound interest on one dollar for that time, and L' the loss on the bridge at the end of the time T', or the amount required to make it as good as new. And let V be the real value of the bridge that would last forever, if all circumstances should remain constant.

"Now, supposing that the money required for building had been borrowed for an indefinite time, the actual expense at the end of the time T to the party paying for the bridge which would last forever would be aV; and the actual expense at the end of the same time for the first bridge, after making it as good as new, would be $aC+L$. These two quantities are equal: therefore the hitherto unknown value of V is

$$C+\frac{L}{a}.$$

"Similarly, at the end of the time T', the expense for the bridge which would last forever would be $a'V$; and that for the second bridge, after making it as good as new,

if the cost had been the real value R, would be $a'R+L'$. As before, these two values are equal; and, therefore,

$$V=R+\frac{L'}{a'}.$$

Equating the two values of V gives

$$C+\frac{L}{a}=R+\frac{L'}{a'},$$

and

$$R=C+\frac{L}{a}-\frac{L'}{a'}.$$

"Now, if the value thus found for R be greater than the cost C', the second bridge is more economical than the first; while, if it be less, the first bridge will be the more economical."

It will be noted that the foregoing method does not mention costs of operation and maintenance. They can be taken into account by adding their capitalized costs to the costs C and C'.

Conditions sometimes arise which render it inadvisable to adopt the most economic of the several compared types of construction—for instance, when the promoters cannot possibly raise the money required to build the kind of structure which they desire, and, consequently, must content themselves, for a time at least, with one that is inferior. An example of this is a proposed railroad through virgin country, where the cheapest kind of a line will serve to develop business, and will suffice for many years to take care of the traffic, although uneconomically as compared with first-class railroads. Under such conditions an engineer possessed of sound financial judgment would advocate building the line at first as cheaply as practicable; adopting comparatively heavy grades, sharp curves, cheap ties, light rails, temporary structures, earth ballast, low-power locomotives, etc.; but paying strict attention to the vital matter of thorough drainage, and studying in advance of construction the question of how the line can be improved later at least expense and without materially interfering with traffic.

As another illustration of this economic consideration, there might be taken the case of a bridge for a crossing where there is danger from washout of falsework. Here it would be advisable to adopt a cantilever structure instead of a layout of simple-truss spans, notwithstanding the fact that it might require considerably more metal and might involve a higher pound price for erection.

Such problems as these may be deemed by some people to be questions of expediency rather than of economics; but the author prefers to treat them as pertaining to the latter, which means that, in the case first mentioned, he would consider it truly economic to build the cheap line and operate it for a while uneconomically rather than to spend at the outset large sums of money in order, later on, to handle economically traffic that possibly might fail ever to materialize; and that in the second case it would

be in the line of true economy to be somewhat extravagant in the cost of superstructure so as to avoid all possibility of disaster during erection.

In this connection it should be noted that it will sometimes be more economic to build a light structure at the outset and later remove it and replace it by a heavier one, rather than to build the heavier one at first, provided that the light structure will serve the purpose adequately for a number of years, and that the replacement can be done without too serious an interruption of traffic. For instance, it might be cheaper to build a $50,000 bridge now and replace it in fifteen years by a bridge costing $100,000 than to build the $100,000 structure at first; for the compound interest on the $50,000 additional investment for fifteen years at six percent amounts to $70,000. It will rarely pay to anticipate traffic requirements by more than twenty or thirty years, in case that thus anticipating them involves a large additional expenditure.

In respect to the economics of design and construction of bridges, the following general suggestions are pertinent:

Anticipating the Future

In all engineering work of both designing and construction, true economy necessitates a thorough consideration of future requirements and possible eventualities, also a provision for meeting the same. For instance, in designing a structure one should consider possible future additions of loading and how to accommodate them; and in construction one should anticipate delays, floods, storms, and other possible difficulties, and should prepare his programme so as to meet them effectively and without any unnecessary expenditure of time, labor, or money. Foresight of this kind is an important element of success in the career of every engineer.

First Cost

It must not be forgotten that under the item "First Cost" must be considered all items of expense, of every kind whatsoever, that may be incurred before the structure is ready for operation. It should include not merely the ordinary estimated cost of construction, or the amount of money to be paid to the contractor, but also promotion expenses, discount due to sale of bonds below par, commissions or other considerations for services rendered, interest on money during construction, right-of-way, rents and minor incidental expenses during construction, administration, legal expenses, engineering, and a proper contingency allowance.

Compensating Factors in Economic Comparisons and Frequent Wide Range of Economic Limits

In the making of economic studies, assumed variations in proportions or types will nearly always increase the cost of certain factors and reduce that

of others, while the values of some factors may remain unchanged. The costs of these various factors tend to balance, so that considerable modifications rarely produce great changes in total cost. This principle is true for all kinds of factors. For instance, in determining the economic span lengths for a truss bridge, increasing the span-length augments the cost per lineal foot for the superstructure and generally, but not always, reduces that of the substructure. Again, in contrasting carbon steel and alloy steel for bridgework, the latter gives smaller weights but greater costs per pound. Also, in comparing railway decks of the ordinary wooden type with ballasted floors, the first cost of the latter is greater, but the expense for its maintenance is less; and this last condition is generally found when pitting steel bridges against reinforced concrete ones.

The effects of variations in factors can be well comprehended by a study of the various diagrams for Chapter XVIII, which treats of the economic span-lengths for simple-truss bridges on various types of foundations at different depths below the elevation of Low Water. The plotted curves show costs per lineal foot for superstructure, substructure, and the total. It will be noted that all substructure curves are concave upward, that the superstructure records are either right lines or easy curves also concave upward, and that the lines for totals are generally very flat curves, concave upward. The economic span-length occurs where the steepness of the substructure curve is just the same as that of the superstructure curve, but it will be noted from the upper of the three curves (the one for total costs per lineal foot) that varying twenty-five feet either way from the absolute minimum augments very slightly, indeed, the total cost per foot, and that a variation therefrom of fifty feet seldom increases the said cost more than two per cent. Furthermore, the exact minimum is dependent somewhat upon the personal equation of the designer; for such matters as sizes of pier bases must be determined largely by judgment, and a slight variation in unit prices of materials in place will move the lowest point of curve some distance horizontally.

From the foregoing it is evidently a waste of time to split hairs when one *knows* he is near the economic point; but, on the other hand, adopting a span-length of 450 feet when 300 feet is the economic limit may sometimes add ten or fifteen per cent to the total cost of structure, consequently one must make sure that his adopted span-lengths do not differ too seriously from those for truly greatest economy.

There is another economic fact that is well worth noting, viz., that whenever in reducing the span-length, some important part reaches minimum size, so that further diminution in length will not reduce that part, it is practically certain that a shorter length will not be economic.

In nearly all economic studies for bridges, the lighter the superimposed load the greater will be the economic length, whether it be span, panel, stringer-spacing, or what-not. This is largely due to the fact that, in case of any design, as the distance in question is reduced, if the loading were

light, certain parts would more quickly reach minimum size than they would if the loading were heavier; and any further material reduction would prove uneconomic.

Systemization

Quoting from "Bridge Engineering," "The systemization of all that one does in connection with his professional work is one of the most important steps that can be taken towards the attainment of success." Moreover, it is one of the fundamental elements of economics in all lines of work.

Time Versus Material

Some designers in their endeavor to save a small amount of material expend a large amount of time, not only of their own but also of other people's, which time when properly evaluated is often greatly in excess of the cost of the material saved. Such economy as this is false; and its practice is unscientific.

Labor Versus Material

Similarly, some designers in an endeavor to cut down quantities in their structures increase the labor thereon to such an extent that the material saved is worth only a small portion of the value of the extra labor expended. For instance, if one were to make a small pier hollow, the concrete thus saved would not be worth anything like as much as the cost of the forms needed to construct the hollow space and that of the reinforcement which would be required in the thin pier walls.

Recording Diagrams

The study of economics is greatly facilitated by the use of diagrams that record quantities of materials, costs of construction, times of operation, etc., for varying conditions. In general, it may be stated that American engineers do not use graphics for studying economics to the extent which is advisable; and that in this they might learn something from their European brethren.

Economics of Mental Effort

Almost nothing concerning this important subject is taught in our technical schools; and but little is known about it by practicing engineers. To be a truly-successful engineer, one has need to study deeply the matter of how best and most economically to utilize his mental forces; how to accomplish the greatest amount of work with the smallest expenditure of effort; how many hours of work per day for long-continued labor will effect the largest accomplishment; to what extent men in various lines of

activity should take vacations, and how these should be spent; what are the effects upon one's working capacity from the use of liquor and tobacco in both small and large quantities; etc. All these are economic questions of great importance; and they need to be given proper attention by every engineer who aspires to efficiency in both himself and his employees.

Again, the development of the faculty of concentration is an economic consideration of the greatest value.

LABOR

The scientific handling of labor is an economic problem of the utmost importance, and a treatise could well be written on the subject. The principal desideratum is to keep the workmen well, happy, and contented; and the best ways to do this are to treat them kindly, make them comfortable, feed and house them well, amuse them in their spare time, avoid working them too long hours, pay them by piece-work when practicable, listen patiently to their complaints, right their wrongs, see that they are well taken care of when they are ill or injured, and evolve, if possible, some feasible method of sharing profits with them. On the other hand, though, drive them hard and continuously during working hours, insist upon their putting in overtime when the conditions truly require it, discharge instantly all insubordinate or otherwise troublesome men, dispense quietly with the services of all shirkers, and insist that everybody put forth his best and most intelligent effort to effect the maximum of accomplishment in the minimum of time.

CHAPTER III

ECONOMICS OF THE PROMOTION OF BRIDGE PROJECTS

In most cases the promoter of a bridge project at the outset is possessed of rather inflated ideas as to the possibilities for gain by the materialization of his proposed enterprise, and, in consequence, is inclined to be uneconomical in his layout of structure and of the money expenditure therefor. If he is to make a success of his venture, he should begin without delay an economic study of his problem; and in this he will require the aid of a bridge specialist of wide experience. It will nearly always be found advisable to keep the first cost of structure down to an absolute minimum, but arranging to increase its capacity from time to time as the augmenting business warrants. Generally this is a necessity, because bankers will not furnish money for an extravagantly conceived proposition; but even if the money be available, it would be bad policy to spend any of it unavoidably at the outset, for the reason that the interest on the extra expenditure, up to the time when the facility in question is really needed, might amount to a large sum of money.

The promoter should investigate thoroughly the possibility for traffic of all kinds, keeping his own counsel about what he is doing, in order to protect himself from hold-up by property owners or the malevolent opposition of possible rival promoters. After finishing this investigation of conditions, he should, if possible, determine the kind or kinds of traffic for which the proposed structure should provide and the probable amounts thereof that there will be, both at the outset and for a long series of years, also the net income it will produce.

The preliminary investigations concerning the probable traffic and other sources of revenue should be made with great care and conservatism. One who is optimistic by nature is prone to overestimate, and no promoter is of any account at all unless he is optimistic; hence he should consider very carefully all uncertain matters connected with the revenue estimates, and should endeavor always to err upon the side of safety. Similarly, in computing the annual cost for maintenance, repairs, and other like expenses, he should be careful to omit no items and to figure each item high enough to be beyond criticism.

After his bridge specialist has reported upon the best type of structure to build, the first cost for the minimum amount of construction at the outset, and the subsequent increased costs for enlargements or betterments, the promotor should complete the economic study of the enterprise and

decide whether it is advisable to undertake the venture. If he experiences any difficulty in making up his mind as to the kinds of traffic for which he ought to provide, his engineer should be able to tell him approximately the cost of structure to carry any kind or combination of kinds thereof; and, knowing the probable receipts therefrom, he should then be able to come to a proper decision.

The question often arises as to whether it will pay to accommodate pedestrian travel; and, in the case of a long structure carrying street cars, it will not. In some cases, however, in order to procure a franchise for building the bridge, it may be necessary to agree to provide footwalks for pedestrians, even if there be very few of them. In the old days of horse-propelled vehicles, it was practicable for pedestrians to cross on the main roadway, but to-day the rapid passage of automobiles would render such a procedure exceedingly hazardous.

In the case of a proposed combined-railway-and-highway bridge, it is often good policy to floor over the railway deck so as to carry temporarily both trains and vehicles, and to provide for attaching brackets in the future to support the latter. Such an arrangement gives very poor service; but it often will suffice for a number of years.

In the building of a railway bridge only, the approaches may be timber trestles, which have a life of ten years, more or less; and at the end of that time they can be replaced either with new timber or by permanent construction. If the bridge be built to carry a double track, and if one track will take care of the traffic for a few years, a single track can be laid on the two inner lines of stringers, and the approaches may be single-track wooden-trestles; or, if preferred, one side only of the double-track structure can be used, and the temporary approaches can be built off-center.

In case of a great scarcity of funds, a double-track bridge can be designed with trusses for half live load and partial dead load, and an arrangement made in advance for the future doubling of trusses. The author evolved and patented this detail many years ago, but has never since had occasion to utilize it in actual construction, although he has estimated upon its employment.

Occasionally it is feasible to build a portion of the main bridge of permanent construction and the remainder of cheap, perishable materials; and this expedient may be in the line of true economy. A quarter of a century ago the author built a bridge across the Missouri River between Council Bluffs, Iowa, and East Omaha, Nebraska, on the basis of part permanent and part temporary construction; and later he replaced the temporary portion with permanent spans. In the original structure the cheapening of everything was carried to the utmost legitimate limit, in order to come within the bankers' appropriation. The pivot pier of the swing span (the largest in the world at that time, viz., 520 feet) was made permanent; but the span itself was stripped of its cantilever brackets for roadways and sidewalks, a portion of the deck was floored over for teams, a single track

was placed at the middle of the double-track space, and pedestrians had to use the roadway. Of course, there could not be railway and highway vehicles simultaneously on the structure, but the roadway was wide enough for teams to pass and for a line of pedestrians on each side of a railroad train. The end piers of the swing span and all the other piers were built of piles and other timbers, the flanking spans were of the combination type, viz., tension members of steel and compression members of timber, and the approaches were single-track timber-trestles for the railway and wooden approaches with fairly-steep grades for the highway. This temporary work was constructed so as to last at least eight years, and it was used for ten, when it was taken out by the author and replaced with permanent construction for the Illinois Central Railroad Company, which had bought the structure for its main-line entrance into the city of Omaha. The temporary work here described was all so well and thoroughly done that, when it was removed, there were no evidences whatsoever of decay or failure. The author is of the opinion that it could have been used for six or eight years longer without the slightest danger of any kind. The evolution of a design such as above described involves economics of the highest type; and the author considers it to be by no means one of his minor achievements in bridge designing and construction.

There is a matter of economic importance which no promoter should ever forget; and that is the growing scarcity of timber, and, consequently, its greater future price. While it may prove temporarily advantageous to use it in his bridge, he should make sure that the metalwork of his superstructure is strong enough and that the foundations of his substructure are sufficiently substantial to carry properly the increased dead load of the spans due to the future substitution of heavy concrete for light timber.

CHAPTER IV

EFFECT ON ECONOMICS FROM VARIATIONS IN MARKET PRICES OF LABOR AND MATERIALS

THE economics of bridge design are not so greatly affected by variations in prices of labor and materials as is commonly supposed, because there is a tendency for all prices to rise and fall more or less uniformly. If they were to do so exactly, the effect on the economics would be absolutely *nil.* It is only when the variations in unit prices for the component materials are irregular that the economics of design are affected, and then in most cases but slightly. It is true that when there is a sudden rise or drop in the cost of superstructure metal erected, the proportionate change in substructure prices will lag behind; but it is generally not long before a state of comparative equilibrium is reached, and the variations in prices become more nearly uniform as compared with those that existed before the change occurred.

So far as economic span-lengths for simple-span bridges are concerned, the practical effect is small indeed; for, even though the theoretic economic length be considerably changed, there is always a rather wide range on either side of the minimum where the costs are but little higher.

Of more importance than this is the effect upon the relative economics of different types of structures, especially those of different materials such as steel and reinforced concrete. In normal times the base price of structural metal fluctuates from about 1.15 cents per pound to about 1.75 cents—it even ran up to 2.25 cents in 1899 and 1900. The cost of fabricated structural metal is subject to still greater changes. Freight rates in different portions of the country vary from about 0.1 cent a pound to over one cent. Local conditions may affect transportation and erection costs materially. For the foregoing reasons the price of structural metal erected is subject to a variation of about two cents a pound, entirely apart from causes influencing the prices of other materials. For instance, in January of 1920, the price of cement was $2.80 a barrel in New York and $2.43 a barrel in San Francisco, while the freight rate on structural metal was 0.27 cent a pound to the former point and 1.25 cents to the latter. At the same time, heavy construction timber cost nearly twice as much in New York as in San Francisco; and while structural iron workers received only 8 per cent more in the former than in the latter, common labor was paid nearly 50 per cent more. Sand, stone, and gravel were considerably cheaper in San Francisco than in New York. At that period, therefore, structural metal was about one cent

a pound cheaper in New York than in San Francisco, whereas concrete and timber were about 40 per cent more expensive in New York.

Again, early in 1915 structural metal erected was let in a few cases as low as three cents a pound, with prices of other materials correspondingly small; while about two years later the said metal erected was for a short time as expensive as ten cents a pound, the prices of other materials showing an increase of less than 50 per cent. In January of 1920, structural metal erected was quoted as low as 7 cents a pound, while other prices were much higher than in 1917. Economic comparisons in 1920 are, therefore, quite similar to those in the pre-war period, while those of 1917 were decidedly different from those of either of the other dates.

The foregoing examples are sufficient to indicate the fact that, when close economic comparisons are to be made, carefully selected unit prices must be used. The statement at the beginning of this chapter to the effect that such variations will seldom radically affect economic comparisons, is nevertheless correct. The result of an economic study made with normally-balanced unit prices will rarely be in error by any serious amount.

This question will be discussed further in the various chapters dealing with comparative economics.

In connection with the elaborate series of computations made by the author in the preparation of his monograph on "Economic Span-Lengths for Simple-Truss Bridges on Various Types of Foundations," he took occasion to figure three sets of economic curves for low-level, double-track, steam-railroad bridges on sand foundations, one with normal unit prices for all materials in place, another for extremely high prices, and the third for extremely low prices. The various unit prices for each case were adjusted according to the author's best judgment, based upon an experience in bridge estimating extending continuously over a longer period of years than he likes to acknowledge. The results of this comparison are given in the following table:

TABLE 4*a*

ECONOMIC SPAN-LENGTHS FOR DOUBLE-TRACK, STEAM-RAILWAY BRIDGES ON SAND FOUNDATIONS AT VARIOUS DEPTHS BELOW EXTREME LOW-WATER

Depth of Foundation below Low-Water	Condition of Material Market		
	Low	Normal	High
100′	290′	275′	275′
150′	330′	310′	325′
200′	375′	360′	375′
250′	425′	430′	425′

The slightness in the variations of these economic span-lengths with the different conditions of the material market is sufficiently evident to warrant

the conclusion that ordinary differences of unit prices will not affect the economic layouts for simple-truss spans having piers on sand foundations. Similarly it might be shown that the conclusion holds good for any other type of pier foundation.

There is, however, a case in the late practice of the author which indicates that abnormally high variations (for the different component materials of bridges) in the changes in unit prices do sometimes affect materially the economics. At the present time the following pound prices for metal erected in suspension bridges prevail:

Wire Cables	23¢ per lb.
Carbon Steel	8¢ per lb.
Nickel Steel	10¢ per lb.

A fair average for the *ante-bellum* unit prices of these materials is respectively 13½, 5, and 7 cents. Using the averages for the two kinds of structural steel as representative market prices, and applying their ratio to the pre-war price of wire cables indicates that a proper present price for the latter in place would be 20 cents. This shows that these cables are about 15 per cent too expensive. As explained in Chapter XIII, the use of existing unit prices instead of *ante bellum* ones raised the span-length of equal cost for highway cantilever and suspension bridges from 1,000 ft. to 1,200 ft., that length for like structures which carry a certain combination of highway and railway loadings from 2,190 ft. to 2,360 ft., and that length for similar bridges subjected to only steam-railway loading from 2,570 ft. to 2,630 ft. These changes are of some importance, hence one must conclude that the economics of certain types of bridges are materially affected by abnormally great variations in the ratios of rise or fall in the unit prices of their component constituents.

The location of a bridge may affect to a certain extent its economic layout, especially for American constructions in foreign countries. For instance, the freight and customs duties on superstructure metal might be very high and the importation of skilled erection-workmen might be a necessity, while the prices of substructure materials and of unskilled labor therefor might be exceedingly low; and with such a combination of conditions the economic span-lengths would be considerably shorter than those governing in the United States.

CHAPTER V

ECONOMICS OF ALLOY STEELS*

WHETHER it be economical or the reverse to use for bridge construction any particular alloy of steel instead of standard carbon steel will depend upon three fundamental conditions, viz.,

A. The ratio of costs per pound erected of carbon steel and the alloy steel under consideration.
B. The ratio of the elastic limits of these two steels.
C. The type and the span length, or span lengths, of the structure contemplated.

If r (greater than unity) is the ratio of costs per pound erected of the alloy steel and carbon steel, and r' (less than unity) is the ratio of elastic limits of the two metals, then for primary truss members of rods or bars the economy in using the alloy will depend upon whether the product of r and r' is less than unity, or mathematically

$$r\,r' < 1. \qquad \text{[Eq. 1]}$$

This criterion will not hold good for spans; because, while the ratio of intensities of working stresses for simple (unstiffened) tension members of untreated steel† is exactly equal to that of the elastic limits, the varying ratios of the intensities of working stresses for compression members and for rigid tension members (on gross sections) differ materially therefrom. Moreover, a certain portion of the weight of metal in a structure is not affected by varying the intensities of working stresses. Again, the criterion would take no cognizance of the reduction of dead load due to the smaller weight of steel involved by using the alloy. The first and second of the variations mentioned are of opposite sign from the third, and, in consequence, the tendency of the combination is to offset; but the preponderance, except in the case of unusually long spans, is in favor of the

* After this chapter was completed, its contents were used as a basis for a paper presented to the *Académie des Sciences* of France, entitled "*L'Émploi Économique des Alliages d'Acier pour la Construction des Ponts*," all quantities in both diagrams and text being changed to French units. The paper was published in condensed form by the Academy on July 12, 1920, and in full by *Le Génie Civil* on July 24, 1920.

† In the case of eye-bars of treated steel, the intensity of working tensile stress is taken as either one-half of the elastic limit or one-third of the ultimate strength—whichever of the two is the smaller.

first two, and hence the criterion for very short spans, such as plate-girder bridges, will be for average cases

$$r\,r'<0.8. \qquad \text{[Eq. 2]}$$

This value really varies from 0.75 for rolled I-beam spans to 0.85 for the longest plate-girder spans.

But in the case of very long spans it is economical to use an alloy when $r\,r'$ is equal to or greater than unity. This is because in such spans the dead load is large in comparison with the live load, even after the latter has been properly increased for the effect of impact; and because, as before indicated, the use of the alloy cuts down the weight of metal in the parts of the structure where it is employed, and thus reduces the total dead load to be carried by the trusses of the span or spans. The greater the span-length the more marked is the economy of adopting alloy steel.

In Figs. 5*a* and 5*b* are given the economic limiting values of $r\,r'$ for simple-span and cantilever steam-railway-bridges. The method of using these diagrams and the formula in Eq. 2 is very simple. It can best be illustrated by a few examples.

Example No. 1

With standard nickel steel erected at 7¢ per lb. and standard carbon steel erected at 5¢ per lb. will it pay to adopt the alloy for plate-girder spans? Here we have

$$r = 7 \div 5$$

$$r' = 35 \div 60$$

$$\text{and } r\,r' = 7/5 \times 35/60 = 0.82$$

This is less than the 0.85 given by Eq. 2 for long plate-girder spans, hence the answer to the question is affirmative.

Example No. 2

Mayarí steel with an elastic limit of 50,000 lbs. per square inch costs 5.5¢ per lb. erected, while carbon steel is worth 4¢. Will it pay to adopt the alloy for building a double-track span of 275 feet? Here we have

$$r = 5.5 \div 4$$

$$r' = 35 \div 50$$

$$\text{and } r\,r' = 5.5/4 \times 35/50 = 0.96$$

Fig. 5*a* gives 0.93 as the limit, consequently the use of that alloy under the market conditions stated would not be economic.

Example No. 3

Standard nickel steel having an elastic limit of 60,000 lbs. per square inch is worth erected 7.3¢ per lb. while carbon steel costs 5.1¢. What are the economics of employing it for building a double-track, simple-truss span of 550 feet? Here we have

$$r = 7.3 \div 5.1$$

$$r' = 35 \div 60$$

$$\text{and } r\,r' = 7.3/5.1 \times 35/60 = 0.835$$

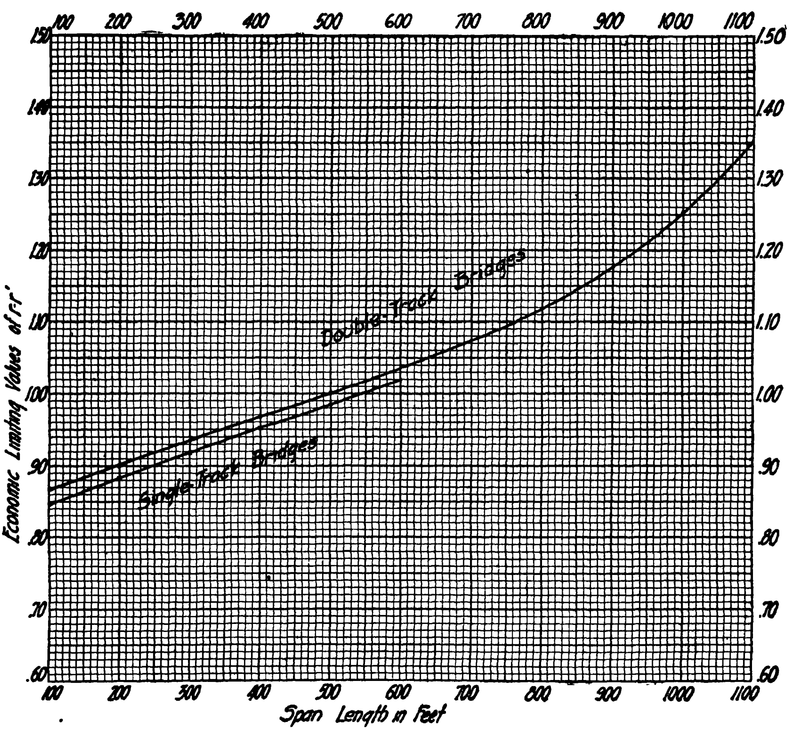

FIG. 5a. Economic Limiting-Values of rr' for Simple-Span, Steam-Railway Bridges.

Fig. 5*a* gives 1.01 as the limit; hence the use of the alloy would effect considerable saving.

Example No. 4

In the investigation for a proposed three-span, cantilever, railway bridge having a main opening of 1,550 feet, it was found that an alloy steel of 75,000 lbs. elastic limit would cost 13.8¢ per lb. erected, while standard carbon

steel erected could be obtained for 6.3¢ per lb. What are the economics of the case? Here we have

$$r = 13.8 \div 6.3$$

$$r' = 35 \div 75$$

$$\text{and } r\,r' = 13.8/6.3 \times 35/75 = 1.02$$

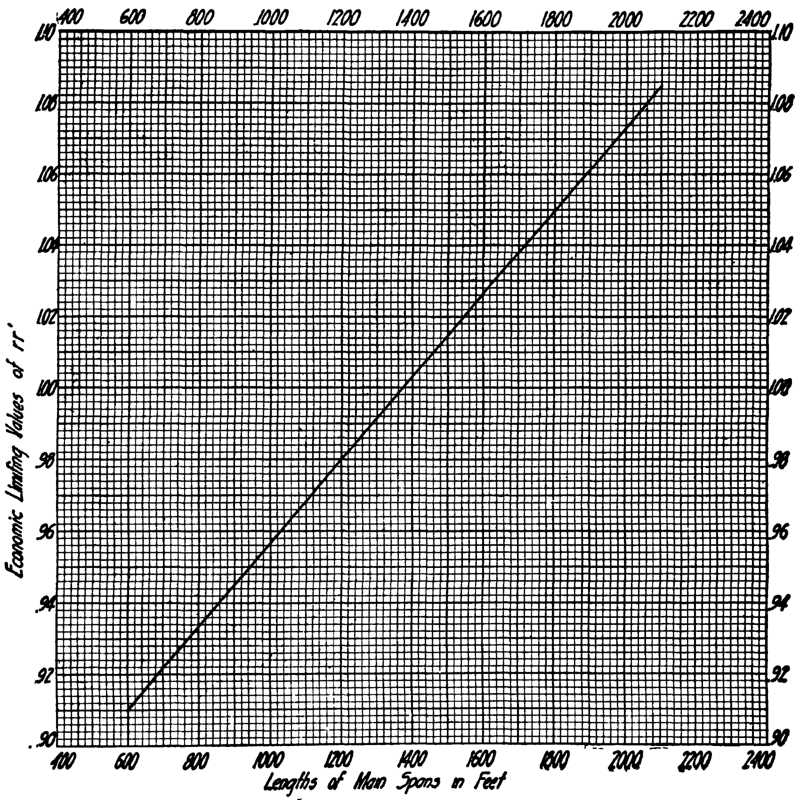

FIG. 5b. Economic Limiting-Values of rr' for Cantilever, Steam-Railway Bridges.

Fig. 5b gives 1.02 as the limit, which shows that the costs of structure are alike for the two steels.

In Figs. 5a and 5b the basis of comparison is standard carbon steel, and the curves were plotted from weights figured for spans actually computed and proportioned. The estimates referred to were those worked out by the author a number of years ago for his two papers, "Nickel Steel for Bridges" and its sequel, "The Possibilities in Bridge Construction by the Use of High Alloy Steels," published by the American Society of Civil Engineers in 1909 and 1915.

In the latter paper the curves of weights of metal for cantilever bridges were extended far beyond the limits of accurate computations by a method specially evolved for the purpose; and, while the said method may not be deemed strictly accurate, it is truly logical and, in all probability, close enough for the economic investigations in the making of which its resulting weights were employed. From the weights plotted in the "speculative zone" of Fig. 7 in that paper, and herein reproduced as Fig. 5*c*, has been prepared Fig. 5*d*, from which can be found the comparative economics of any procurable or practically-possible high-alloy steels for cantilever structures having main spans exceeding the longest yet constructed, viz., the 1,800 ft. span of the Quebec Bridge.

The following example will illustrate its use:

Example No. 5

What are the comparative economics of standard nickel steel costing in place 8.5 cents per pound and an alloy steel having an elastic limit of 80,000 lbs. per square inch and costing in place 11.2 cents per pound, for a three-span, cantilever bridge which has a main opening of 2,450 feet?

From Fig. 5*d* we find the comparing ratios of weights, in relation to a hypothetical steel having an elastic limit of 100,000 lbs. per square inch, to be 1.23 and 1.66; hence, compared with each other, the ratio of average weights per foot for the two materials will be $1.23 \div 1.66 = 0.74$. The ratio of pound prices erected is $11.2 \div 8.5 = 1.318$. The product of these ratios is $1.318 \times 0.74 = 0.98$. As this is less than unity, the high-alloy steel is more economic than the standard nickel steel, and the saving involved is about two per cent.

It may be noticed that in this last investigation there is an implied assumption to the effect that the steel for the structures is unmixed, or, in other words, that carbon steel is not employed for light members or minor parts. The explanation for this is four-fold.

First. In such long spans it pays to cut out every possible pound of dead load.

Second. Everything connected with the structure being on a stupendous scale, there will be no members so light as to be proportioned for rigidity and not for strength.

Third. In the detailing of heavy members of alloy steel, it will generally be advisable to use the alloy so as to make the details themselves as strong as possible.

Fourth. Even if there were a small amount of carbon steel employed in these phenomenally long and heavy bridges, the percentage thereof in any two compared cases of alloy steels of different elastic limits would be so nearly alike that the reliability of Fig. 5*d* would not be affected.

A question has been raised as to the accuracy of the curves in Figs. 5*a* and 5*b*, on the plea that the relative amounts of nickel steel and carbon

steel which should be used in a nickel-steel span depend upon the ratio of unit costs and that of elastic limits, and are not constant. Thus, if the excess unit cost of nickel steel were large, it might pay to employ carbon

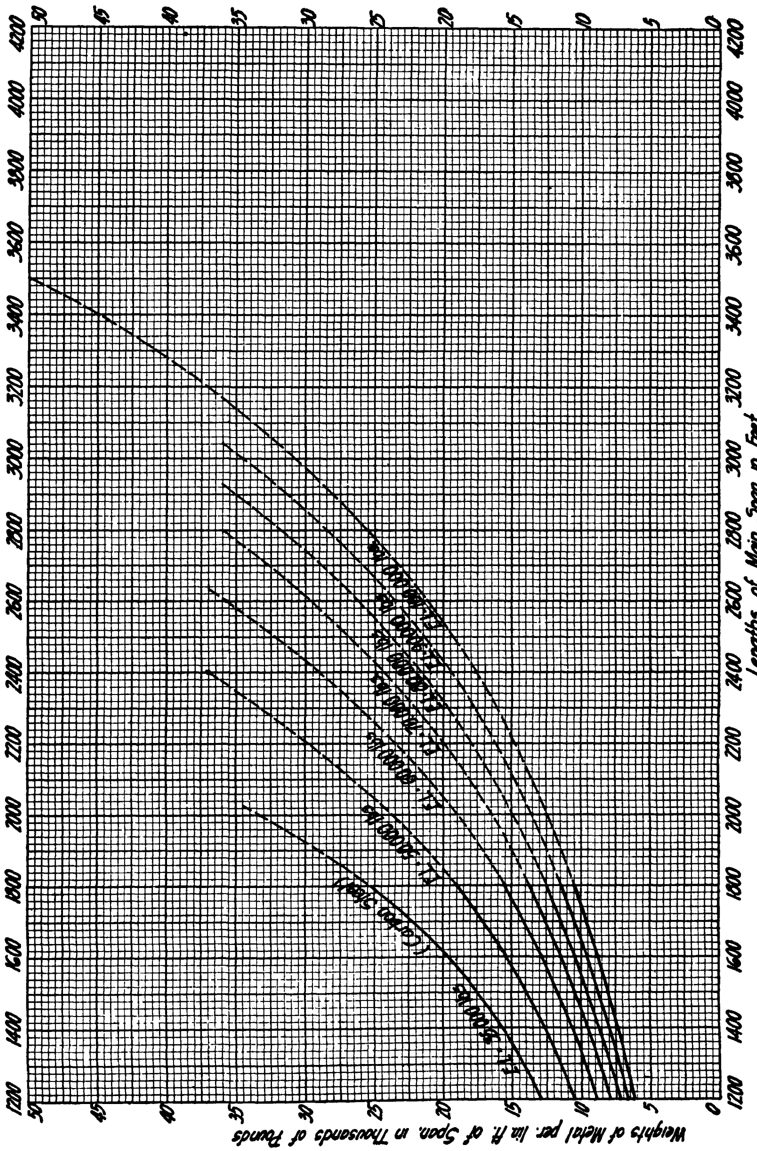

FIG. 5c. Total Weights of Metal per Lineal Foot of Span for Double-Track, Steam-Railway, Cantilever Bridges of Carbon Steel and Alloy-Steels of Various Elastic Limits.

steel in the floor system, and even in the posts; while, if the said excess were small, it might be economical to use the alloy in those parts. The author's reply to this criticism is that his experience in figuring with alloy

steels leads him to believe that when they are really worth considering for any case, the stronger metal should be used for the floor system so as to lighten the dead load, even if *per se* that portion of the structure were made slightly more costly, and that carbon steel should be substituted for the alloy steel only in such places where, without increasing the sectional

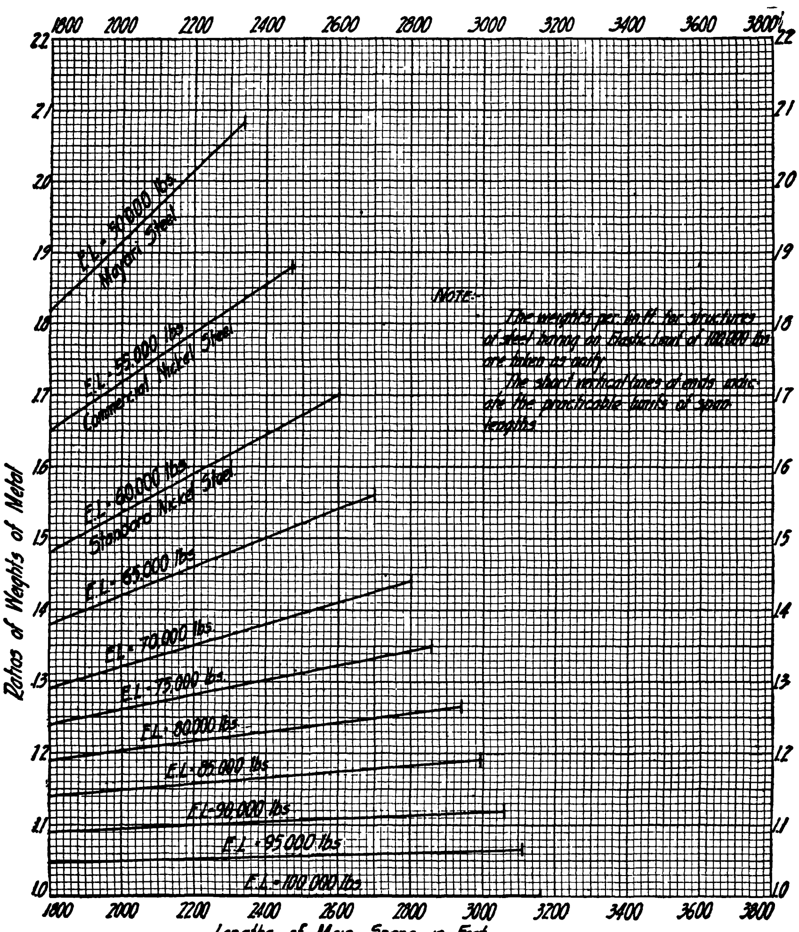

FIG. 5*d*. Diagram Showing Comparative Economics of All Procurable, or Practically-Possible, High-Alloy Steels for Long-Span, Cantilever Bridges.

area, the weaker metal would provide ample strength—for instance, in the lateral systems and lighter posts of moderately-light, short-span bridges.

It was upon this assumption that the author many years ago accumulated the data upon which, substantially, the diagrams given in this chapter were prepared; and under like conditions they may be relied upon abso-

lutely. But if there should arise a case in which the choice of carbon steel or alloy steel for the floor system is debatable, the said diagrams might be considered as only approximately correct; and in such a case some special computations of weights and costs of metal might become necessary.

Attention is called to the phenomenally short time required for the solution of any economic problem in the use of alloy steels for bridgework by means of Figs. 5*a*, 5*b*, and 5*d* when the elastic limits and the pound prices erected of the steels to be contrasted are known. Hereafter it will be unnecessary for anyone desirous of employing an alloy of steel in the design of a bridge to wade through the two papers previously mentioned or the author's other writings on the subject of alloy steels, because the economic results of all his previous investigations are concentrated into the three diagrams last indicated.

The use of alloy steel in bridgework is only in its infancy, for thus far there have been very few bridges built of it. In 1903 the author began his economic study of the question of "Nickel Steel for Bridges," and the investigation required more than three years to complete. He found that good, reliable bridge-steel could be manufactured with an elastic limit of 60,000 lbs. per square inch by adding to the usual charge of molten metal $3\frac{1}{2}\%$ of nickel; and as a result of his findings several large bridges were constructed of that alloy. A few years later the great demand for nickel in the manufacture of armor plate for ships-of-war enhanced the price of that metal to such an extent as to make it too expensive to employ in bridge construction; and the advent of the Great War in 1914 sent the price soaring. Although the cessation of hostilities has decreased the demand for the alloying metal, its price is still unsettled and probably has not yet been sufficiently reduced to warrant its employment for bridges—nor in fact, has there been any call of late for metallic bridges of importance. Just as soon, though, as the general business of the country attains once more a sound condition, there will be a request for some large bridges of long span, because a number of them are even now being seriously considered; and, when that time arrives, the question of alloy steels for such structures will become a paramount issue, and either nickel or some other suitable alloying agent or agents for strengthening bridge metal will be greatly in demand.

Since the time when nickel became too expensive to use in bridges, several alloy steels, other than nickel steel, have been either exploited or suggested, the principal ones being Mayarí steel, purified steel manufactured by the electro-metallurgical process, aluminum steel, vanadium steel, and silicon steel. On account of the great cost of nickel and the other alloying metals, there is a tendency on the part of a few American bridge specialists to employ high-carbon steel in important constructions. In the author's opinion, this is a dangerous policy to adopt, because high-carbon steel is brittle and, therefore, unsuitable for bridgework. He has never been willing to use it in any of his constructions, notwithstanding the fact that the specifications of his "De Pontibus," written in 1897, per-

mitted its employment in certain of the larger members of long fixed-spans, but barred it entirely from movable spans.

In the specifications of "Bridge Engineering," written in 1915, no high steel is permitted, excepting only a certain grade of metal termed "machinery steel," for which the limit of reduction of area is 35% and that of the elongation in two inches is 18%, both of which values are somewhat greater than those specified for high steel in "De Pontibus."

Mayarí steel is a natural alloy of nickel-chromium steel, containing from 1% to 1.5% of nickel and generally from 0.2% to 0.75% of chromium (although occasionally the proportion of this last element runs considerably greater), with sulphur below 0.04%, phosphorus below 0.03%, and manganese as desired. The ore comes from a deposit of some 25,000 acres at Mayarí in the province of Oriente on the Island of Cuba. On account of the large irregularities in the elastic limit of Mayarí steel, it is not deemed safe to count upon more than 50,000 lbs., but as the nickel and chromium which exist in the ore cost no more than the iron, the actual cost of manufactured bridge members really ought to be about the same as for carbon steel, unless it be that the content of these foreign elements has to be increased. It may be that Mayarí steel will prove to be the basis of the future ideal alloy of steel for long-span bridges; but it is more likely that the irregularity of composition of the smelted metal will render its employment for that purpose too objectionable.

Thus far there has been no systematic attempt to use for bridgework the "purified steel" manufactured by the electro-metallurgical process, the main objection to it being that up to the present time it has never been produced in large melts or on a grand scale, as would be necessary if it were employed in steel structures.

As for aluminum steel, it has never even been in the running, although advocated for bridgework by a few engineers who apparently were not properly posted concerning its properties.

At one time the author had the hope that vanadium steel might solve the problem of alloy steel for bridgework; but from all he can learn of late about that alloy it appears to fall short in several essential requirements.

Silicon steel in bridgework has been tried, and with satisfactory results. It is about as difficult to manufacture as other alloy steels, the elastic limit being forty-five thousand pounds per square inch. It has not been very much used as yet, but those who have tried it seem satisfied with the results. It ought not to be very expensive per pound, as the alloying material is not costly.

Of late the element molybdenum has been looming up as a possibility in the solution of the high-alloy, bridge-steel problem, but thus far no experiments with it have been made looking towards its use in bridge construction. The author nowadays is indulging in a "pipe-dream" about what he has dubbed "Nichromol" steel, a prospective alloy of nickel, chromium, and molybdenum, with an excess of manganese above the amount

ordinarily used in steel-making, as being the ultimate solution of the said problem. He is endeavoring to make the dream come true by trying to induce a combination of miners, metallurgists, and steel manufacturers to furnish the requisite money for an elaborate series of experiments to find an ideal high-alloy of steel for long-span bridges; and, perhaps, if he lives long enough, he will be successful. He feels confident that within three years after actually starting the investigation, with ordinarily good luck in respect to governing conditions, and with a reasonable expenditure of money, he could find how to manufacture the material desired at a fairly-moderate pound-price. A successful solution of this problem would be epoch-making in respect to the economics of bridgework.

There appeared in the New York *Times* of February 18th, 1920, a notice concerning some experiments that are being made in France upon the production of high-grade steels by a modification of the Bessemer process. If these experiments prove to be successful, the manufacture of the author's proposed "Nichromol Steel" may be readily materialized. The following is the notice referred to:

PARIS, Feb. 16.—A revolution in the steel industry is promised by four inventors who are working here. Final tests of their process are now being made. Their claim is that hard steel—nickel, chromium, manganese and the other kinds—can be manufactured at roughly the same cost as ordinary Bessemer steel, with the sole added expense of the alloys involved

In a mill on the northern outskirts of Paris to-day five experiments were made each involving the production of a ton and a half of high-class steel. There was little in appearance to distinguish the new from the ordinary Bessemer process. There was an ordinary furnace, packed with coal and iron. The metal was fused at a relatively low temperature and then passed to a furnace, where the temperature was raised to 1500° C. and the impurities burned out. By the Bessemer process a relatively small percentage of impurities, chiefly phosphorus and sulphur, is eliminated. The result is that Bessemer steel is suitable for only ordinary work and cannot be employed as raw material for the high grade steel necessary for many phases of industry.

The essential feature of these experiments is that by the addition of certain secret substances and by means of a certain undivulged process the ordinary Bessemer steel process can be applied to produce steel as pure as that derived electrically.

These results are predicted by the inventors:

First, France will be in a position to produce high-grade steel at the same cost, or approximately the same cost, as ordinary steel plus the expense of the alloys, while special steel containing no alloys can be produced at the same price as ordinary steel.

Second, high-grade steel, which hitherto could not be employed for such ordinary purposes as railway rails, etc., now becomes available for the everyday purposes of commerce.

The four inventors have been working for more than six months and have satisfied themselves that the high-grade steel they produce answers to every test, whether of chemical analysis or of physical properties, such as hardness, tensile stress, malleability, etc.

One of the inventors is Jules Lambrecht of Herstal, Belgium, who worked as a steel expert for France during the war. Another is Marc Antoine, also a Belgian, known as an authority on railroad steel. The other two, whose names may not now be mentioned, are Frenchmen.

To-day's tests were attended by a number of steel experts and scientists who took

away samples of the inventors' product. The presence of these men attested the seriousness of the experiments, and it is reported that the inventors may have discovered a process as valuable as that of Sir Robert Hadfield, which is reserved for Government use in England, despite the demand on the part of private enterprise. The French inventors have no connection with the Government.

They call attention to the fact that ordinary Bessemer steel, because of the impurities of the metal, wears badly and irregularly. Hard steel having a much longer life can be made to bear the same strains with very much less content of material, and thus the country that is able to produce high-grade steel at the cost of ordinary steel will benefit by the immensely increased output and will be able, because of superior methods, to compete on very favorable terms with Great Britain, the United States and other steel-manufacturing countries.

At a time when the whole world is in need of steel of every sort, the cheapening of the highest grades and the reduction in the amount of labor required to produce it are factors of capital importance in the general work of reconstruction.

It is pointed out that, while the electrical process for producing pure steel is excellent, it requires an enormous expenditure of energy and labor, and is consequently extremely costly. The Bessemer process, on the other hand, while relatively cheap, has hitherto failed to remove impurities. What the inventors say they can do is to obtain pure steel by the use of the Bessemer process with slight alterations.

Since the preceding was written the author has secured some information concerning molybdenum steel for which he has been searching during the last year or two, and which had been refused him by a high authority in the employ of an automobile manufacturing company—possibly with the thought that, if molybdenum were adopted for bridgework, there would not be a large enough supply left for the use of the automobile industry. Some six months ago, however, the author was so fortunate as to secure from the president of the Climax Molybdenum Company of New York and Colorado certain interesting general information concerning the alloying properties of molybdenum, with the promise of detailed data as soon as they could be collected and formulated for publication in pamphlet form. In accordance with that promise, there came to hand a short time ago an advance copy of a booklet entitled, "Molybdenum Commercial Steels," issued as a trade catalogue by the before-mentioned company. The work contains a mass of detailed information about the alloy; and, although the data apply directly to steel for automobiles, it has proved practicable to make from the tabular matter deductions indicating how the said alloy might be applied to bridge construction. It certainly contains sufficient statistics to enable an investigator to draft a programme of studies and tests for determining the best practicable combination or combinations of molybdenum and other alloying elements with iron in order to produce the high alloy of steel for bridgework for which the author has been searching these many years.

While it is true that the publication is unquestionably a trade catalogue for the promotion of the sale of molybdenum, it is pointed out therein that, in order to avoid any undue rose-coloring caused by the natural tendency in writing such a work to "put one's best foot foremost," the MS. was submitted for comment to the officials and metallurgists of some

of the largest alloy-steel manufacturers and consumers in the United States, with the result that it met with their approval—their experience with the various molybdenum-steel types described conforming to the statements, facts, and figures set forth. Moreover, the general treatment of the subject gives *prima facie* evidence of a spirit of fairness; and the style of work is that of a technical scientist and not that of a promoter.

Unfortunately, the records all deal with heat-treated steel, which, while applicable for eye-bars, is not suitable for built members of bridges; but by inquiry from the writer of the pamphlet the author obtained a small amount of data concerning tests of one of the steels untreated, as well as some results of tests of carbon-molybdenum steel that were made on an accidental melt from which chromium had unintentionally been omitted. From the same source it was learned that the proportionate increase in ultimate strength and elastic limit due to the addition of molybdenum is practically the same for untreated as for treated steels.

From the contents of the pamphlet, the additional information just mentioned, and the author's previous experience with nickel steel, the deductions which follow have been drawn.

For simplification of the discussion, and in order to distinguish readily between the various combinations of alloying materials, the author has taken the liberty of evolving and using the following nomenclature:

Carmol = Carbon-Molybdenum.
Chromol = Chromium-Molybdenum.
Nichromol = Nickel-Chromium-Molybdenum.
Nicmol = Nickel-Molybdenum.
Chrovanmol = Chromium-Vanadium-Molybdenum.
Nichro = Chrome-Nickel.
Chrovan = Chrome-Vanadium.

In order to utilize the diagrams of this chapter for finding the economics of molybdenum as an alloying material for bridge steel, an understanding will have to be arrived at, in order to determine properly the values of r'; because, while a high intensity of working stress may be employed for heat-treated eye-bars, a much lower one will have to be adopted for the untreated built-members; and, again, it would never be legitimate to use a working stress greater than one third of the ultimate strength. In heat-treated steels the elastic limit generally falls but little below the ultimate strength, hence it would not do to use one half of its amount for the working tensile stress as is done in the case of untreated steel.

As the data concerning the untreated molybdenum steel are rather meagre, it will be necessary to make a few approximate assumptions in determining the elastic limits and intensities of working stresses. For instance, one is that the proportion of untreated and treated steel in a pin-connected bridge will be about as two is to one. This is fairly accurate, and will suffice for a preliminary study of which the sole purpose is to indi-

cate in outline the final investigation it will be necessary to make, in order to establish the suitability of molybdenum steel for bridgework, determine the best proportions for the alloying elements, and demonstrate the economics of the alloy in comparison with other bridge steels, both plain and alloyed.

From "Molybdenum Commercial Steels" and the before-mentioned supplementary data furnished to the author the following tables have been copied or prepared:

Heat-Treated Chrome Steel with and without Molybdenum

TABLE 5a

TENSILE TEST ON CHROME STEEL

Analysis

Carbon	Manganese	Chromium	Molybdenum
0 27	0 63	0 99	None

Physical Properties. (1 inch round)

Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent
130,000	139,000	16 5	58

TABLE 5b

TENSILE TEST ON CHROMOL STEEL

Analysis

Carbon	Manganese	Chromium	Molybdenum
0.26	0.64	0 76	0 31

Physical Properties. (1 inch round)

Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent
142,000	151,000	18.5	62

HEAT-TREATED CHROME NICKEL STEEL WITH AND WITHOUT MOLYBDENUM

TABLE 5c

TENSILE AND DYNAMIC TESTS ON NICHRO STEEL

Analysis

Carbon	Manganese	Silicon	Chromium	Nickel	Molybdenum
0 33 av.	0 5 av.	0 18 av.	1 0 av.	3 27 av.	None

Physical Properties. (Rolled Bars 1" to 2")

Elastic Limit	Tensile Strength	Elongation Per Cent.	Red. of Area Per Cent	Izod Ft.-Lbs.	Brinell Hardness
116,700	135,200	19 6	57 1	61	270

TABLE 5d

TENSILE AND DYNAMIC TESTS ON NICHROMOL STEEL

Analysis

Carbon	Manganese	Silicon	Chromium	Nickel	Molybdenum
0 27 av.	0 6 av.	0 3 av.	0 86 av.	2 95 av.	0.43 av.

Physical Properties. (On finished shaft)

Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent	Izod Ft.-Lbs.	Brinell Hardness
130,000	142,000	20.5	65	67	303

HEAT-TREATED CHROME-VANADIUM STEEL WITH AND WITHOUT MOLYBDENUM

TABLE 5e

TENSILE TESTS ON CHROVAN STEEL

Analysis

Carbon	Manganese	Chromium	Vanadium	Molybdenum
0 36 av.	0 5 av.	0 9 av.	over 0.16	None

Physical Properties. (1″ round)

Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent	Brinell Hardness
146,500	167,500	16	54 5	340

TABLE 5f

TENSILE TESTS ON CHROVANMOL STEEL

Analysis

Carbon	Manganese	Chromium	Vanadium	Molybdenum
0 39	0 48	1 06	0 17	0 85

Physical Properties. (1⅛″ round)

Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent	Brinell Hardness
170,000	190,000	19	60	392

Heat-Treated Nickel Steel with and without Molybdenum

TABLE 5g

Tensile Tests on Nickel Steel

Analysis

Carbon	Manganese	Nickel	Molybdenum
0 30	0.25	4.00	None

Physical Properties

Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent
108,000	120,000	16	55

TABLE 5h

Tensile Tests on Nicmol Steel

Analysis

Carbon	Manganese	Silicon	Nickel	Molybdenum
0 33	0 25	0 18	4 50	0 58

Physical Properties. ($1\frac{1}{8}''$ round)

Oil Quench	Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent
1450° F.	164,600	173,800	16 0	53 4
1500° F.	166,500	176,000	15 5	55 3
1550° F.	165,100	175,000	15 5	54 0
1600° F.	164,900	173,300	15 5	55 6
1650° F.	166,000	174,400	15 0	55 0
	165,000 av.	175,000 av.	15 5 av.	54.7 av.

Heat-Treated Carbon Steel with Molybdenum

TABLE 5*i*

Tensile Tests on Carmol Steel

Test No. 1

Analysis

Carbon	Manganese	Molybdenum
0.24	0 69	0 32

Physical Properties

Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent
92,000	114,000	20	66.5

TABLE 5*j*

Test No. 2

Analysis

Carbon	Manganese	Molybdenum
0 17	0 49	0 96

Physical Properties

Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent
102,000	135,300	19.5	61

Untreated and Treated Chrome Steel with Molybdenum

TABLE 5k

Tensile Tests on Chromol Steel (Untreated)

Analysis

Carbon	Manganese	Chromium	Molybdenum
0.26	0.65	0.95	0.32

Physical Properties

Diam. of Test Piece	Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent
7/8"	89,700	116,900	18 5	55
1¼"	104,000	115,000	17.5	54
2"	75,000	100,000	20.5	60

TABLE 5l

Tensile Tests on Chromol Steel (Treated)

Analysis

Diam. of Test Piece	Carbon	Manganese	Chromium	Molybdenum
7/8"	0.26	0 65	0 95	0 32

Physical Properties

Drawing Temperature	Elastic Limit	Tensile Strength	Elongation Per Cent	Red. of Area Per Cent
500° F.	220,000	232,000	12	48
1100° F.	130,000	142,000	20	63

In making cost estimates for alloy-steel bridges, the following unit prices have been assumed. They are based upon the market conditions governing in March, 1920.

Ordinary carbon-steel work, erected........	8¢ per lb.
Nickel................................	40¢ per lb.
Chromium..............................	22¢ per lb.
Vanadium residue in steel (to include wastage)...........................	$10.00 per lb.
Molybdenum...........................	$ 2.50 per lb.
Heat-treatment of eye-bars	1¢ per lb.

There will first be determined from the various diagrams in "Molybdenum Commercial Steels" the best drawing temperature for the heat treatment of alloy steels to be used in bridgework. In modern alloy-steel practice there has come into vogue the term "Quality Number," meaning the product of the elastic limit in pounds per square inch by the reduction of area expressed in ratio to unity. This is considered the criterion of excellence, because the maximum values of these two quantities are desirable. As the drawing temperature is increased, the elastic limit drops and the reduction of area augments, consequently that temperature which makes the product of the two values a maximum probably gives the best result for combined strength and toughness.

An analysis of fifteen diagrams in "Molybdenum Commercial Steels" shows that the average best drawing temperature is 800° F., it being a trifle greater for oil quenching than for water quenching. A higher drawing temperature than this seems to be preferable for automobile steel, in order to secure great toughness for resisting shock; but it gives very good results with steel having main characteristics suitable for bridge building, the average percentage of elongation for small test pieces being 15.5. As the principal standard bridge specifications call for an elongation of 15 per cent in specimen tests of nickel steel, it may be concluded that the molybdenum-steel alloys above considered are satisfactory in respect to this criterion.

There will next be determined, as accurately as the rather meagre data available will permit, for all of the usual kinds of alloy steels, the economic benefit to be derived by adding molybdenum to the other ingredients of the steel.

Carmol Steel versus Carbon Steel

From a reliable source the author has learned that the heat-treatment of carbon-steel eye-bars raises the elastic limit from 30,000 lbs. per square inch to at least 50,000 lbs. per square inch, and that the present-day cost of the heat-treatment is about one cent per pound.

Let us assume that in a pin-connected bridge the average value in place

for carbon steel, both treated and untreated, is 8.5¢ per pound, then referring to Table 5*i*, we shall have the following:

CARMOL STEEL

99.68 lbs. steel	@	8.5¢	= $8.47
0.32 lb. molybdenum	@	$2.53	= .81
100.00 lbs. alloy	@	9.28¢	= $9.28
Excess cost in manufacture and erection, say	@	1.00¢	= 1.00
		Total	= $10.28

$$\therefore r=\frac{10.28}{8.5}=1.21$$

Working tensile stress for heat-treated steel ... = 38,000 lbs.
Ditto untreated (assumed) ... = 30,000 lbs.
Average = $\frac{1}{3}$ (2×30,000+38,000) ... = 32,700 lbs.

CARBON STEEL

Working tensile stress for heat-treated steel .... = 26,700 lbs.
Ditto untreated ... = 16,000 lbs.
Average = $\frac{1}{3}$ (2×16,000+26,700) ... = 19,600 lbs.

$$\therefore r'=\frac{19,600}{32,700}=0.6$$

$$\text{and } r\,r'=1.21\times0.6=0.726$$

As this is far lower than any values of $r\,r'$ given in Figs. 5*a* and 5*b*, it is evident that the addition of molybdenum to carbon steel will always effect a large economy.

Chromol Steel versus Chrome Steel

Referring to Tables 5*a* and 5*b*, we have the following:

CHROMOL STEEL

98.93 lbs. Steel	@	8.5¢	= $8.41
0.76 lb. Chromium	@	25¢	= .19
0.31 lb. Molybdenum	@	$2.53	= .78
100.00 lbs. Alloy	@	9.38¢	= $9.38

CHROME STEEL

99 lbs. Steel........................	@ 8.5¢	=$8.42
1 lb. Chromium....................	@ 25¢	= .25
100 lbs. Alloy........................	@ 8.67¢	=$8.67

$$\therefore\ r=\frac{9.38}{8.67}=1.082$$

CHROMOL STEEL

Working tensile stress for heat-treated steel.... =50,300 lbs.
Ditto untreated (approximately)............. =39,000 lbs.
Average $=\frac{1}{3}$ $(2\times 39{,}000+50{,}300)$.............. =42,800 lbs.

CHROME STEEL

Working tensile stress for heat-treated steel.... =46,300 lbs.
Ditto untreated (assumed)................... =33,000 lbs.
Average $=\frac{1}{3}$ $(2\times 33{,}000+46{,}300)$.............. =37,400 lbs.

$$\therefore\ r'=\frac{37{,}400}{42{,}800}=0.87$$

$$\text{and } r\,r'=1.082\times 0.87=0.94$$

As this is lower than nearly all the values of $r\,r'$ in Figs. 5*a* and 5*b*, it may be concluded that it is almost always economic to add molybdenum to chrome steel; or, in other words, chromol steel is preferable to chrome steel for bridgework on account of both cost and toughness.

Nicmol Steel versus Nickel Steel

Unfortunately, the records in "Molybdenum Commercial Steels" do not give any tests for the untreated Nickel Steel; but as the author once made some tests on a $4\frac{1}{4}\%$ nickel steel for eye-bars, he will use the results thereof in this crude investigation.

Referring to Table 5*h*, and employing averages, we have the following:

NICMOL STEEL

94.92 lbs. Steel....................	@ 8.5¢	= $8.07
4.50 lbs. Nickel..................	@ 43¢	= 1.94
0.58 lb. Molybdenum..............	@ $2.53	= 1.47
100.00 lbs. Alloy....................	@ 11.48¢	=$11.48

Referring to Table 5*g*, we have

NICKEL STEEL

96.00 lbs. Steel	@ 8.5¢	=	\$8.16
4.00 lbs. Nickel	@ 43¢	=	1.72
100.00 lbs. Alloy	@ 9.88¢	=	\$9.88

$$\therefore r=\frac{11.48}{9.88}=1.162$$

NICMOL STEEL

Working tensile stress for heat-treated steel.... =58,300 lbs.
Ditto untreated (approximately)............. =45,000 lbs.
Average $= \frac{1}{3}$ (2×45,000+58,300)............. =49,400 lbs

NICKEL STEEL

Working tensile stress for heat-treated steel.... =40,000 lbs
Ditto untreated........................ =28,000 lbs.
Average $= \frac{1}{3}$ (2×28,000+40,000).............. =32,000 lbs.

$$\therefore r'=\frac{32,000}{49,400}=0.647$$

$$\text{and } r\,r'=1.162\times0.647=0.752$$

As this is lower than any of the values of $r\,r'$ given in Figs. 5*a* and 5*b*, it is evident that the addition of molybdenum to nickel steel will always effect a large economy.

NICHROMOL STEEL VERSUS NICHRO STEEL

Referring to Tables 5*c* and 5*d*, we have the following:

NICHROMOL STEEL

95.76 lbs. Steel	@ 8.5¢	=	\$8.14
0.86 lb. Chromium	@ 25¢	=	0.22
2.95 lbs. Nickel	@ 43¢	=	1.27
0.43 lbs. Molybdenum	@ \$2.53	=	1.09
100.00 lbs. Alloy	@ 10.72¢	=	\$10.72

NICHRO STEEL

95.7 lbs. Steel	@ 8.5¢	=	$8.13
1.0 lb. Chromium	@ 25¢	=	0.25
3.3 lbs. Nickel	@ 43¢	=	1.42
100.0 lbs. Alloy	@ 9.8¢	=	$9.80

$$\therefore\ r=\frac{10.72}{9.8}=1.094$$

NICHROMOL STEEL

Working tensile stress for heat-treated steel.... =47,300 lbs.
Ditto untreated (approximately)... .... =37,500 lbs.
Average $=\frac{1}{3}$ (2×37,500+47,300).............. =40,800 lbs.

NICHRO STEEL

Working tensile stress for heat-treated steel.... =45,000 lbs.
Ditto untreated (approximately).............. =35,500 lbs.
Average $=\frac{1}{3}$ (2×35,500+45,000).............. =38,700 lbs.

$$\therefore\ r'=\frac{38{,}700}{40{,}800}=0.949$$

$$\text{and } r\,r'=1.094\times0.949=1.038$$

Referring to Figs. 5*a* and 5*b*, it is seen that the simple-span length corresponding to this product is about 600 feet and the main opening for a Type A cantilever is about 1,600 feet; and as most spans are below this limit, it appears probable that there is seldom any economy in adding molybdenum to chrome-nickel steel for the manufacture of bridges; but the data used for this investigation are so crudely approximate that this conclusion requires corroboration. It would need some elaborate and expensive experimenting to determine this economic point with accuracy. The addition of the molybdenum, though, would undoubtedly improve the quality of the alloy by increasing its resistance to shock.

CHROVANMOL STEEL VERSUS CHROVAN STEEL

Referring to Tables 5*e* and 5*f*, we have the following:

CHROVANMOL STEEL

97.92 lbs. Steel	@ 8.5¢	=	$8.32
1.06 lbs. Chromium	@ 25¢	=	0.27
0.17 lb. Vanadium	@ $10.00	=	1.70
0.85 lb. Molybdenum	@ $2.53	=	2.15
100.00 lbs. Alloy	@ 12.44¢	=	$12.44

CHROVAN STEEL

98.94 lbs. Steel....................	@ 8.5¢ =	$8.41
0.90 lb. Chromium................	@ 25¢ =	0.23
0.16 lb. Vanadium.................	@ $10.00=	1.60
100.00 lbs. Alloy.....................	@ 10.24¢=	$10.24

$$\therefore\ r = \frac{12.44}{10.24} = 1.215$$

CHROVANMOL STEEL

Working tensile stress for heat-treated steel.... = 63,300 lbs.
Ditto untreated (entirely assumed)............ = 47,000 lbs.
Average = $\frac{1}{2}$ $(2 \times 47{,}000 + 63{,}300)$.............. = 52,400 lbs.

CHROVAN STEEL

Working tensile stress for heat-treated steel.... = 56,000 lbs.
Ditto untreated (entirely assumed)........... = 41,500 lbs.
Average = $\frac{1}{3}$ $(2 \times 41{,}500 + 56{,}000)$.............. = 46,300 lbs.

$$\therefore\ r' = \frac{46{,}300}{52{,}400} = 0.884$$

$$\text{and } r\,r' = 1.215 \times 0.884 = 1.074$$

From Figs. 5*a* and 5*b* we find that this product corresponds to a simple-span length of about 750 feet and to a main cantilever opening of about 2000 feet; and, as these are excessive, it may be concluded, for bridgework, that there is no advantage in adding molybdenum to chrovan steel.

The next economic question to solve is that of the gain involved by increasing the percentage of molybdenum in an alloy of steel. From certain diagrams in "Molybdenum Commercial Steels" the following data for comparison of chromol steel of Classes *A* and *C* have been excerpted, the treatment-temperature being 800° F.

CLASS A

Average analysis, Carb. 0.15, Mang. 0.38,
Chrom. 0.72, Moly. 0.28
E. L. = 105,000 lbs., Ult. = 130,000 lbs., Elong. = 20%, Red. = 65%.

CLASS C

Average analysis, Carb. 0.18, Mang. 0.37,
Chrom. 1.05, Moly., 0.73,
E. L. = 150,000 lbs., Ult. = 170,000 lbs., Elong. = 18%, Red. = 58%.

CLASS A

99.00 lbs. Steel	@ 8.5¢	=	$8.42
0.72 lb. Chromium	@ 25¢	=	0.18
0.28 lb. Molybdenum	@ $2.53	=	0.71
100.00 lbs. Alloy	@ 9.31¢	=	$9.31

CLASS C

98.22 lbs. Steel	@ 8.5¢	=	$8.35
1.05 lbs. Chromium	@ 25¢	=	0.26
0.73 lb. Molybdenum	@ $2.53	=	1.85
100.00 lbs. Alloy	@ 10.46¢	=	$10.46

$$\therefore r = \frac{10.46}{9.31} = 1.124$$

CLASS A

Working tensile stress for heat-treated steel.... = 43,300 lbs.
Ditto untreated (assumed)........................ = 37,000 lbs.
Average = $\frac{1}{3}$ (2×37,000+43,300).............. = 39,100 lbs.

CLASS C

Working tensile stress for heat-treated steel.... = 56,700 lbs.
Ditto untreated (assumed)........................ = 48,500 lbs.
Average = $\frac{1}{3}$ (2×48,500+56,700).............. = 51,200 lbs.

$$\therefore r' = \frac{39{,}100}{51{,}200} = 0.763$$

$$\text{and } r\,r' = 1.124 \times 0.763 = 0.858$$

From Figs. 5*a* and 5*b* we find that this is lower than those recorded there for very short spans, from which it is evident that increasing the molybdenum content up to three quarters of one per cent is in the line of economy, provided that the resulting alloy be not too brittle. The writer of "Molybdenum Commercial Steels" states that there is an advantage in using molybdenum up to a limit of one per cent; but whether this can

be done with impunity in the case of bridge metal can only be proved by actual experiments with the high alloy thus produced.

COMPILATION

The following table gives a résumé of the findings and deductions herein evolved:

TABLE 5*m*

Kind of Steel	Cost per Lb. in Place	Average Intensity for Tension
Carmol.	9 28¢	32,700 lbs.
Chromol.	9 38¢	42,800 lbs.
Nicmol.	11 48¢	49,400 lbs.
Nichromol. . . .	10 72¢	40,800 lbs.
Chrovanmol . .	12 44¢	52,400 lbs.

An inspection of Table 5*m* shows that the Carmol and Nichromol steels are out of the running; hence it will be necessary to test the three others. Comparing Chromol and Nicmol, we have:

$$r\,r' = \frac{11.48}{9.38} \times \frac{42,800}{49,400} = 1.06$$

From Figs. 5*a* and 5*b* we find that this product corresponds to a simple-span length of about 700 feet and to a main cantilever opening of about 1,900 feet, hence it may be concluded that, except for unusually long spans, Nicmol Steel is not as economic as Chromol steel.

Comparing Chromol and Chrovanmol, we have:

$$r\,r' = \frac{12.44}{9.38} \times \frac{42,800}{52,400} = 1.083$$

As in the last case, this is so great as to show that Chromol steel is more economic than Chrovanmol steel.

From all that precedes it is evident that the most promising combination of alloying materials for a high-grade bridge-steel is one of chromium and molybdenum; and a good analysis to test as a starter would be as follows:

Carbon	0.25
Manganese	0.75
Chromium	0.75
Molybdenum	0.75

It would not do, however, to confine one's attention solely to Chromol steel in making a systematic study of the question of the best high-alloy of steel for bridgework, because the preceding study has been made upon data some of which, of necessity, are so roughly approximate that the findings therefrom have to be taken *cum grano salis*. All that can properly be claimed for this tentative investigation is that it will serve as an indication that an exhaustive series of tests on molybdenum steel for bridges would be well worth while, and that it suggests about what might be expected as a result thereof.

CHAPTER VI

COMPARATIVE ECONOMICS OF BRIDGES AND TUNNELS

THE contrasting of a bridge with a tunnel or a combination of tunnels for any proposed crossing, in respect to the question of economics, is by no means easy; because, in order to make a perfectly just comparison, the facilities afforded by the two structures should be alike. Generally, laymen make the mistake of pitting a single-track tunnel against either a double-track bridge or a purely railway tunnel against a combined-railway-and-highway bridge. As it is usually uneconomic to make the interior diameter of a tunnel tube much greater than twenty feet, because the cost of such a tube increases very rapidly with the diameter, and as any projected bridge that is in competition with a tunnel is necessarily a structure of some importance, and, therefore, wide of deck, it is evident that the comparison will probably always be made between one bridge and two or more tunnels. Again, as the travel in a tube of twenty-foot internal diameter must either be in one direction only or else very slow, two tubes will be required in order to obtain rapid transit. The reason for this is that, in a tube just large enough for two lines of traffic in opposite directions, the speed is absolutely limited to that of the most-slowly-moving vehicle; because it would be impracticable for a rapidly-moving automobile to turn out so as to pass a slow vehicle without facing the line of traffic coming in the opposite direction; and a single breakdown would quickly block all motion. Anyone who gives the subject any thought must quickly arrive at the conclusion that the motion of traffic in any highway-tunnel tube must be restricted to one direction only.

Another obstacle to the satisfactory comparison of these fundamentally-different types of transportation structures is the as-yet-unsolved problem of ventilation. Many engineers contend that it is absolutely unsafe to run automobiles through a long tunnel because of the exceedingly-poisonous carbon-monoxide given forth during combustion; while others say they are certain that such traffic can be maintained without danger. Probably this question will be settled finally during the next five years, and in the only convincing manner possible, viz., by building a long tunnel for automobile travel and operating it. As it is stated by medical men of high authority that the action of carbon-monoxide upon the human system is cumulative, it may prove difficult, expensive, or even totally impracticable to dilute the poison to such an extent as to make the atmos-

phere in the tube perfectly safe for breathing, especially by people of feeble constitution.

There is another factor that is likely to have considerable influence in deciding between bridge and tunnel, viz., that driving over the former is much more agreeable than driving through the latter. Again, walking through a long tunnel is so unpleasant that it would seldom be worth while to make any provision whatsoever for pedestrian travel therein.

Then, too, the difference between the height of climb over a bridge and the depth of descent into a tunnel will have some effect upon the choice of structure. In general, it may be stated that the dip of a tunnel is greater than the rise of a low-level bridge, about equal to that of an ordinary high-level bridge, and less than that of a structure below which pass ocean-going vessels.

Supposing, however, that all the pros and cons of the two types, barring cost only, about balance each other, the question for settlement would be one of economics; and then the decision would favor the bridge, because *for equal facilities* the cost of the tunnel almost always greatly exceeds that of the bridge. Of course, the conditions affecting the latter might be so onerous as to increase the cost of the structure beyond any reasonable limit—for instance a span of unprecedented length; but, in general, the bridge costs less than the tunnel.

In the author's practice he has twice had occasion to pit bridge against tunnel. In the first case a highway suspension bridge with a span of 1,750 feet and a total clear roadway, including sidewalks, of 96 feet was contrasted with two tubes which together gave a clear roadway of 56 feet. The result was that the tubes, in spite of their smaller carrying capacity, cost about fifty per cent more than the bridge.

In the other case, a single-track-railway tube cost considerably more than a double-track, low-level bridge—and even a little more than a low-level, combined-railway-and-highway, double-deck structure. Comparing the tunnel and a high-level bridge with a clearance above high water of 150 feet, the ratio of costs of a two-tube, railway tunnel and a double-track, railway bridge was 1.1; and comparing a three-tube tunnel for both railway and highway traffic with a combined-railway-and-highway, high-level bridge, in which comparison the facilities afforded were nearly equal, the ratio of costs was 1.12—both of these results being in favor of the bridge.

There is one advantage which, theoretically, the tunnel possesses over the bridge; and under certain conditions it might become practically operative, viz., that, while a wide-decked bridge has to be built all at once, several tunnels of the same aggregate width can be constructed from time to time as the traffic necessitates, thus saving for some years the interest on the difference between first costs.

A claim has been made that any bridge having more than about forty feet of deck-width will congest the traffic to such an extent that there is no advantage to be obtained from the extra width above the said forty feet;

but such an idea is a fallacy, because the incoming and the outgoing vehicles could enter and leave the bridge at points two blocks apart, and a double-track, electric-railway line could enter and leave by the street between. If this arrangement would not separate the incoming and the outgoing traffic sufficiently, the entrances and exits might be located four, six, or eight blocks apart—in fact there need be no restriction as to the total width of deck in case that this method of traffic separation be adopted.

By employing a spiral approach of large diameter, the traffic could leave the periphery thereof at three, or even more, points, thus making the approaches to the said spiral of different lengths, but all comparatively short.

Some months after the preceding was written with the intention of considering the treatment of the subject as closed, the author had occasion to prepare for the American Society of Civil Engineers a paper entitled "Bridge *versus* Tunnel for the Proposed Hudson River Crossing at New York City"; and as it gives much additional information upon the general economic question involved in this chapter, it is here reproduced practically *verbatim:*

Whilst making lately some extensive calculations concerning the costs and economics of long-span suspension-bridges for his forthcoming treatise on "Economics of Bridgework," the author has had occasion to figure weights of metal for a number of such spans; and by means of the resulting data he was able to undertake an investigation of the comparative costs and efficiencies of bridges and tunnels for the long-talked-of crossing of the North River at New York City. Thinking that the present is an auspicious time for a thorough discussion of the subject, he has collected and condensed the results of his labors and incorporated them in this memoir for the Society.

For some years he has been of the opinion that the best and most economic solution of the problem under consideration is to carry all street cars and subway cars beneath the water and the strictly-highway traffic above it. As far as the question of desirability is concerned, this arrangement would be the best practicable for the following reasons:

First. In respect to cost of operation, the tunnel would require a dip of ninety feet below high-water, and the bridge a rise of one hundred and eighty feet above it; consequently it is evident that, as far as the matter of expenditure of energy is concerned, the tunnel would be decidedly preferable. The difference in cost of power would be very apparent to the management of the electric railways, and possibly also to the operators of heavy trucks, but it would not be noticed at all by the owners of automobiles used mainly for pleasure traffic. When an automobilist is about to climb a long, heavy grade, he seldom thinks anything concerning how much extra his gasoline is going to cost him; but the officers of an electric railway line generally figure with the greatest of care on the item of power expense, and aim to reduce it to a minimum.

Second. In regard to the difference in the expenditures of time in climbing up and down the approaches of the two crossings under comparison, the gravity of this matter would be duly appreciated by the railroad company and more or less by the operators of trucks, but it would not be recognized by automobile owners and users.

Third. As to the agreeableness of the two kinds of crossing, while people do not particularly fancy going under ground before they are ultimately compelled to, they soon become accustomed to passing beneath the water in electric cars, as is evidenced by the many New York business men and women who reside on Long Island or in New Jersey. Perhaps in time the drivers of trucks would become so used to traversing tunnels as not to object to the gloom that is inherent in such passage; but the general public, almost to a man (and certainly to a woman), would always greatly prefer driving over a structure that provides good air and light, and usually a fine view of the harbor and the surrounding country, in comparison with traversing a long, cramped, and dingy tube.

Fourth. In relation to the question of sanitation, there is practically no greater danger to health in passing through a tunnel in which the power used is always electrical than there is in traversing a bridge; but the safe ventilation of a tube carrying automobile traffic is as yet an unsolved problem. Figures show that such ventilation, if feasible, would be exceedingly expensive, and the velocity of the passing air would be excessive.

Again, as carbon monoxide, like arsenic, is a cumulative poison, there exists a possibility that the regular daily passage through a tube where the gas remains constantly, even in minute quantities, would eventually undermine one's health. Besides, there is always the chance of a blockade of traffic with the tunnel full of automobiles, and these may be counted upon to discharge more or less products of combustion even when standing still. Such a blockade might result in a holocaust.

The author sees no serious objection, however, to building the contemplated highway tunnels under the North River; because if, after completion, they prove to be unsafe, or otherwise unsatisfactory for automobile traffic, they can either be used by electric railway cars or else a moving platform can be put in to carry vehicles through without letting them use their own power. The experiment of automobile transportation through long tunnels might be worth making, for the results in any event would prove of great interest and value to the engineering profession, as well as to the general public.

In making the comparative estimates of cost of bridges and tunnels for this crossing, the author adopted four (4) per cent grades on the approaches of both structures and clear roadways of twenty-two feet, with sidewalks eleven feet wide. He utilized as a basis for his comparison the cost estimates for tunnels given in the report of Clifford M. Holland, M. Am. Soc. C. E., Chief Engineer of the New York State Bridge and Tunnel Commission and the New Jersey Interstate Bridge and Tunnel Commission; but

it was necessary to modify some of the items for the increased diameter of tube and the steeper approach grades, also to omit the costs of equipment and administration, as these are not included in the bridge estimates. For the modification of cost due to a changed diameter of tube, it was assumed that the cost per lineal foot increases and decreases as the square of the exterior diameter. This assumption is almost exactly correct, because the thickness of the shell varies directly with the diameter, as does also the length of the periphery, consequently, the volume of the shell is approximately proportional to the square of the diameter. Again, the volume of excavation for any shield-driven, circular tunnel varies as the square of the diameter; and as almost the entire cost per lineal foot is that of the shell plus that of the excavation, it is evident that the assumption mentioned is justified.

Increasing the exterior diameter of the tube to thirty-one (31) feet makes the interior diameter twenty-seven (27) feet. This is just enough to provide a clear roadway of twenty-two (22) feet in the highway tunnel and allows just enough space for two lines of the widest subway cars in the electric-railway tunnel.

In the latter type it is feasible to contrast the cost of a double-track tube with that of two single-track tubes; but in a highway tunnel it is not, because a breakdown of a single vehicle would block all traffic until it is hauled out by sending a double-ender wrecking-car into the tube in a reverse direction to that of the traffic. With such an occurrence in a double-track tunnel, the automobiles could pass by the wrecked vehicle.

If there were an accidental stoppage of an electric railway car in a single-track tube, it would cause no more obstruction to rail traffic than it would if the accident had taken place in a double-track tube that operates in both directions.

The author computed all the quantities of materials in substructure, superstructure, and approaches of six structures, in order to plot the two cost curves for bridges shown in Fig. 6a. On that diagram are given the total costs for bridges with their approaches and for tunnels with their approaches based upon the unit prices which governed at the time the tunnel estimates were prepared. The principal ones of these are as follows:

Wire cables in place	23¢ per lb.
Nickel steel in place	11¢ per lb.
Plain concrete in shafts and anchorages	$16.00 per cu. yd.
Mass of pneumatic bases	$35.00 per cu. yd.

In proportioning the substructures the author made the dimensions as small as considerations of true efficiency would permit, and did not attempt any beautification of structure by an unnecessary enlargement of piers. If these are properly built to meet all possible conditions of loading and so as to provide against future deterioration caused by the elements, that ought to suffice; and so doing should be deemed good engineering practice.

In order to make the investigation general instead of particular, so as to apply to all other similar river crossings by very-high-level suspension-bridges, the span-lengths were assumed to be 1,500, 2,300, and 3,000 feet, to permit the plotting of curves of total costs for both bridges and tunnels with their approaches for various widths of river. In this special case the span length would have to be about 2,900 feet; and in all cases the length

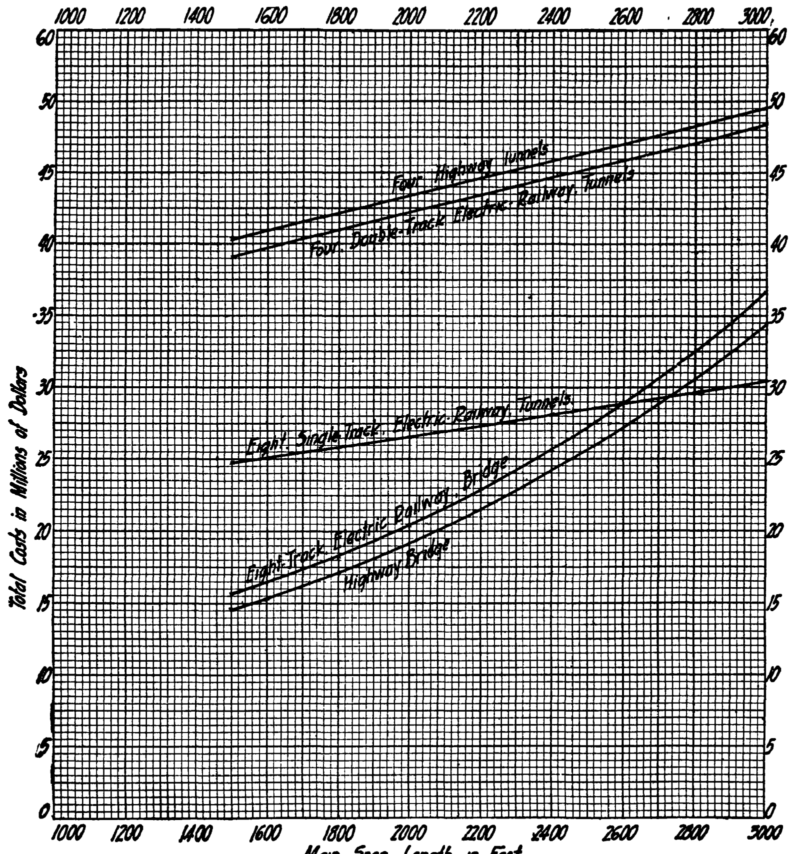

FIG. 6a. Diagram of Total Costs of Highway and Electric-Railway Bridges and Tunnels, with their Approaches, for Crossings Similar to that of the North River at New York City.

of the horizontal portion of the tunnel has been assumed to be exactly equal to that of the main span of the competing bridge.

In the highway-structure comparison there were adopted for the bridge three clear roadways of twenty-two feet each, and two sidewalks of eleven feet each, corresponding to four double-track tubes each of twenty-two feet clear roadway. The driveways were assumed to consist of creosoted-

wooden-block pavement supported by reinforced-concrete slab, and the sidewalks to be of reinforced granitoid.

In the electric-railway-structure comparison, the floors were assumed to be of the open type, having wooden ties and guardrails and the usual steel rails; and eight lines of track were adopted so as to compare with four lines of double-track tubes and with eight lines of single-track tubes.

In every possible manner the comparison was made fair and equitable, excepting that the costs of right-of-way and property damages for the approaches, for obvious reasons, had to be ignored. This militated against the tunnel; hence in any actual case of comparison an allowance would have to be made for the difference in costs of right-of-way and property damages for the approaches to the two structures. In the case of the bridge, by purchasing a large city-block close to the water and building a spiral approach thereon, the cost of the said right-of-way and property damages would be reduced to a minimum; and even that cost might be offset by constructing a high office building above the spiral. Such a building, owing to its location, ought to possess a high rental value.

Fig. 6*a* shows the total costs of main span, piers, and approaches for both highway and electric railway bridges and those of the corresponding tunnels.

As before indicated, the shortest span-length that can be used for the proposed North River bridge is about 2,900 feet, for which length the diagram gives approximately the following costs:

Highway Bridge	$32,500,000
Four Highway Tunnels	$49,000,000
Electric-Railway Bridge	$34,500,000
Four Double-Track, Electric-Railway Tunnels	$48,000,000
Eight Single-Track, Electric Railway Tunnels	$30,000,000

This indicates that there is a saving of cost in favor of the highway bridge amounting to $16,500,000, and one of $4,500,000, in favor of single-track, electric-railway tunnels. The former saving is far greater than the difference in costs of right-of-way and property damages for bridge and tunnel is ever likely to be; hence the conclusion is reached that, for the proposed North River crossing, it is not only better from every point of view, but also *more economic* to carry automobile traffic by a bridge and electric trains by single-track tunnels.

The question arises as to how cheaply there could be built at present prices a combination of bridge and tunnels to care for both kinds of traffic. For a number of years two single-track tubes would take care of the electric-railway trains, hence the cost would be as follows:

Highway Bridge	$32,500,000
Two Single-Track Tunnels	7,500,000
Total	$40,000,000

If it were decided that a bridge having a total width of roadways of forty-four (44) feet and two eleven-foot sidewalks inside the trusses, would suffice for possible future conditions of traffic, the amount of money required for the combination would reduce to about $33,500,000.

It must not be forgotten that these totals do not include any allowance for right-of-way, property damages, equipment, or administration.

In conclusion the author offers the suggestion that, in view of the facts presented in this memoir, it might be advisable to give the economics of the proposed crossing of the North River some further study before finally committing the community to the policy that is now contemplated.

The preceding paper was delivered to the American Society of Civil Engineers in May, 1920, was accepted by the Publication Committee, and was printed immediately, advance copies of it being distributed for discussion. It was slated for delivery at the meeting of September third; but, upon very short notice, its reading was indefinitely postponed. The author hopes that, in the interests of engineering economics, this injunction against a thorough discussion of an important engineering problem of great magnitude will not prove to be permanent.

For about thirty years there has been discussed in the public press the proposed construction of an immense bridge across the North River to carry all kinds of traffic, including steam-railway trains; and to-day there is serious talk of materializing the project by building for that purpose a structure to cost two hundred millions of dollars. In the author's opinion, the construction of a high bridge over the Hudson at New York City for the purpose of transferring freight and passenger trains would involve a serious economic blunder from the engineering standpoint. His reasons for this rather drastic and sweeping statement are as follows:

First. It would cost more to build at this location a standard railway bridge carrying n tracks than it would to construct n single-track tunnels.

Second. The right-of-way and property damages would be much greater for a railway bridge than for the corresponding tunnels.

Third. The cost of operation to cover rise and fall is twice as great for the bridge as for the tunnels.

While the unnecessary expenditure of a large sum of money for the construction of the proposed bridge might be pardoned, it would be exceedingly uneconomic to saddle for centuries to come upon posterity a financial burden that will involve needlessly lifting and lowering ninety feet a load of two tons for each ton of freight carried across the river.

CHAPTER VII

COMPARATIVE ECONOMICS OF HIGH-LEVEL AND LOW-LEVEL CROSSINGS

THE term high-level is applied to a structure having all its spans fixed and its deck at a considerable elevation above high-water level; but the term low-level is generally interpreted as applying to structures having one or more movable spans, although, strictly speaking, a bridge over a non-navigable stream may be a low-level one without having any movable span.

The comparison between a proposed high-level and a proposed low-level crossing for any stream is generally more dependent upon the condition of expediency than it is upon that of economy. The convenience of the passengers is often the criterion that will settle the question; but sometimes convenience has to be ignored because of the paramount condition of first cost. Again, the comparison will depend largely upon how much higher the high-level bridge must be than the low-level one, and upon the height and slope of the banks. The clear channel required will also be of some importance, the wider the channel the greater the advantage for the high-level bridge.

Sometimes in a long, high-level structure with only one channel span required, it is practicable to build all the others on grade and as deck spans, economizing on substructure and shortening the approaches, thus lowering materially the total cost of the high-level bridge.

The advantages and disadvantages of a low-level bridge over a high-level one are as follows:

ADVANTAGES

a. The first cost is almost always less.
b. The costs of maintenance and repairs are less.
c. The entrance and exit are closer to the river.
d. The climb is less, and, therefore, the total amount of power of all kinds required for the climb is less.
e. The time spent in crossing is less.

DISADVANTAGES

f. Increased annual expense due to the several items of cost in connection with operating the movable span.

g. Interruptions to travel over the structure from opening the movable span.

h. A somewhat greater obstruction of the thoroughfare in respect to vessel traffic as well as to the passage of the water, in case of the adoption of a swing span with its protection.

A thorough consideration of all these criteria will have to be made before a logical conclusion can be reached as to which type of layout is preferable.

The correct ratio of first costs for a low-level bridge and a high-level one of the same capacity and strength at any proposed crossing must, of course, be determined from layouts, computations of quantities of materials, and estimates based upon current prices of the said materials and of labor. It is sometimes necessary, however, to make a hurried economic comparison; and as an aid in so doing the author has established the following results, based upon assumed conditions that may properly be deemed average or normal.

Let us assume a double-track-railway crossing of a river like the Missouri, requiring 1,500 feet of main spans with a vertical clearance of 50-feet above high water for passing vessels, a variation of twenty-five feet between extreme stages of water, a horizontal bed-rock 75 feet below high-water elevation, one bank high and sloping back from high-water line at a ratio of three horizontal to one vertical and the other bank level and very low; and let the approach grades be one per cent. Under such conditions the deepest water will nearly always be found comparatively close to the higher bank, and the position of channel will be permanent; consequently it would be legitimate to employ a single through span over the second opening from the high-bank end.

With certain assumed medium unit prices for materials in place, the minimum costs of these two structures proved to be as follows:

Low-Level Bridge	$1,204,000
High-Level Bridge	$1,530,000

The ratio of these costs is 1.27

With a low, flat approach on each side, and the other conditions unchanged, it would be necessary in the case of the high-level bridge to adopt a level grade over the entire river so as to provide against any drastic shifting of channel; and in the case of the low-level bridge to figure on being able to take down the towers, machinery, and house and shift them to any one of the other openings. Under these conditions the costs of the two structures proved to be as follows:

Low-Level Bridge	$1,256,000
High-Level Bridge	$2,604,000

The ratio of these costs is 2.08

For one of the author's standard-type highway-bridges, having four per cent grades on the approaches and sand foundations at a depth of 125 feet below high water, the other conditions being as previously indicated for the crossing with one high and one low bank, the estimated costs of structure were as follows:

Low-Level Bridge	$1,386,000
High-Level Bridge	$1,371,000

The ratio of these costs is 0.99.

For the layout with two low banks the costs of the two structures were as follows:

Low-Level Bridge	$1,426,000
High-Level Bridge	$1,623,000

The ratio of these costs is 1.14.

The main reason for high-level structures costing more than low-level ones is the greater lengths and costs of the approaches; and as in highway bridges the grades thereon are much steeper than on railway bridges, their ratios of costs of high-level and low-level structures are less.

It will be necessary to compute carefully for each type the total annual costs of maintenance, repairs, sinking fund, and operation, capitalize these totals, and add the results to the estimates of first cost. A comparison of these sums will determine the *financial* economics of the two types compared.

To be strictly accurate, however, in determining these comparing figures, one should estimate the total annual costs of all kinds of power expended in climbing the approaches, capitalize these, and add them to the previous sums before making the comparison; but, as considerable guess-work would be involved in making such a computation, it might be exact enough for all practical purposes to assume that the total annual costs of power are the same for the two types of structure.

CHAPTER VIII

COMPARATIVE ECONOMICS OF STEEL AND REINFORCED-CONCRETE STRUCTURES

THE settlement of the question as to which costs more, a steel bridge or a reinforced-concrete one of the same capacity, is a difficult task. For structures of ordinary span-lengths, under normal conditions of the material market, the first cost of the steel bridge is the smaller, but when the price of that metal takes a sudden jump the reverse is true. However, in a short time either the prices of concrete materials rise to correspond or else the cost of the metal gradually drops back until a normal ratio of unit prices for bridge materials once more exists.

But the question of the economics of the two types is not settled by a comparison of first costs alone, because the elements of maintenance and repairs must be considered; and these are much greater for the steel structure than for the reinforced-concrete one. The latter requires no painting, and there should be no renewal of parts called for, excepting only the pavements, while in the former these items are often large, especially that of painting, if the upkeep be properly performed. It is necessary, therefore, to capitalize the said items and add the results to the first costs, in which case under normal conditions there is generally but little difference; and as the painting of steel structures is very likely to be neglected and the metal, in consequence, to lose its normal areas, it will often pay to adopt the reinforced-concrete construction, even when the economic comparison indicates a slight disadvantage by so doing.

The fact that the annual cost of maintenance and repairs is less for reinforced-concrete bridges than for steel ones tends to render the former the more popular, especially amongst those persons who do not make a practice of comparing values strictly upon the basis of the principles of true economy. Such persons, too, are prone to say that the reinforced-concrete structure is superior to the steel one because of its longer life, not recognizing the facts that a properly designed, built, and cared-for steel bridge will do its work for a very long term of years, and that the longevity of reinforced-concrete structures as yet is a matter of surmise, in view of the uncertainty about the efficiency of the concrete in protecting the reinforcing steel perpetually against rusting—which, if allowed to continue, will quickly and inevitably disintegrate the said concrete and destroy the structure.

The comparison of the types is complicated also by the matter of personal equation in designing; for, with the most elaborate and rigid specifications ever written, two computers working independently on the same job would be likely to vary in their final results far more in reinforced-concrete work than they would in steelwork. This is due partially to the fact that the science of reinforced-concrete bridge designing is newer and less highly developed than that of proportioning steel bridges, and also because in the former type equal strength can be secured with varying proportions of concrete and steel. As the most economical proportions are frequently not known, it is evident that the combined costs of the two materials in place may differ appreciably. Again, in arch bridges there is often a choice of ratio of rise to span or even of span-lengths; and as the effect on economics by variations in these features is not yet determined with accuracy, the final estimates of cost made by the two computers are liable to be still more widely divergent on this account.

The amount of attention paid to æsthetics when making the design generally affects the cost of a concrete bridge more than it does that of a steel one; hence this factor has to be given consideration when contrasting the two types in respect to the matter of economics.

The size of the live load, too, is likely to affect the comparison, because a diminution thereof cuts down the cost of a steel structure much more than it does that of a concrete one.

For short-span bridges, reinforced-concrete has an advantage over steel in respect to rigidity of structure; and under certain conditions such an advantage may be of importance, but ordinarily it is not.

There is another complication of the question, which, however, ought not to be allowed to exist, viz., the fact that many small reinforced-concrete bridges are designed by inexperienced and incompetent computers, who are hired by municipalities on account of the low compensation they are willing to accept.

Still another cause for divergence is the great variation in costs of excavation for foundations at different localities. This affects the substructure costs for reinforced-concrete arch-bridges far more than it does those for the corresponding steel structures, because of the larger footings and shafts of the former.

Is it then impracticable for a bridge engineer to settle quickly the comparative economics of the two types of construction for a proposed bridge? Not at all—only it will take more time than that required for the determination of most of the economic problems dealt with in this treatise. The method to be followed is this: Having obtained in advance all the unit prices for materials, figure the cost of the steel structure by means of the diagrams of quantities given in Chapters LV and LVI of "Bridge Engineering," then find that for the reinforced-concrete bridge by employing the rules, tables, and diagrams given for that purpose in the latter chapter, fixing by judgment, whenever necessary, an allowance for difference in

costs of excavation. As explained in Chapter XVII of this work, the labor involved in making such computations as the above-mentioned is by no means onerous.

The greater the span-lengths which are necessitated as minimum by the conditions of the crossing, the more favorable is it to the steel structure in the economic comparison; and when a certain span-length has been reached, the reinforced-concrete structure becomes impracticable. What this limiting span-length is, designers have not yet determined with general satisfaction; but the author is of the opinion that it is not very far from seventy (70) feet for girders and three hundred (300) feet for arches. While it is practicable to build reinforced-concrete arch-spans of greater length than the latter figure, much trouble would be involved during their erection by the unequal and abnormally great settlement of the falsework, which settlement tends to distort the arch rings and, in consequence, to give the structure an unsightly appearance. Such settlement can be reduced, if sufficient care be taken; but an excessive amount of the latter adds to the time required for fieldwork and, consequently, to the first cost.

In general, it may be stated that a high-level crossing is usually more favorable to a steel structure than to one of reinforced-concrete, as are likewise deep foundations and perilous erection conditions; also that, other things being equal, a great width of deck generally militates in favor of the reinforced-concrete type of construction; as do, too, the remoteness of the site from the source of the structural-steel supply and the cheapness of common labor available for fieldwork.

It may be that some reader of this chapter will claim that it is rather indefinite in its conclusions and deals mainly with glittering generalities. Possibly there may be some justice in such a claim; but it must not be forgotten that the economics of the two contrasted types of bridges is primarily dependent upon such arbitrary and uncertain conditions as the relative prices of steel and cement, the availability of concrete aggregate, the comparative costs of high-grade and low-grade labor, and the distances of the bridge site from the various sources of supplies. This fact makes it impossible to come to any fixed or reliable conclusion concerning the relative economics of steel and reinforced-concrete bridges.

CHAPTER IX

COMPARATIVE ECONOMICS OF DIFFERENT TYPES OF ORDINARY STEEL STRUCTURES

UNDER the term ordinary steel structures will be included only girder or simple-truss fixed spans of the following types:

Deck, rolled-I-beam spans.
Deck, plate-girder spans.
Half-through, plate-girder spans.
Deck, open-webbed, riveted spans.
Through, riveted spans.
Deck, pin-connected spans.
Through, pin-connected spans.

The economics of all other types of steel bridges will be treated in other chapters.

ROLLED-I-BEAM SPANS

These are always deck structures because of the shortness of their limiting lengths. They are preferable to plate-girder bridges up to the limit which due consideration for the question of deflection sets, even if the economic limit for weight of metal be exceeded. This is because of their extreme simplicity for both manufacture and erection, and also at most times because the base prices for rolled I-beams are lower than those for plates and light shapes—besides the cost of shopwork is less, owing to the small amount of detailing and the comparatively few rivets that have to be driven. In respect to the economics of erection, they are generally built of standard lengths and sizes, thus permitting the larger railroad systems to keep a supply of them in stock to be used for emergencies. Moreover, they are put in place very readily, and there are but few rivets to drive. Of course, plate-girder spans could be standardized and employed in the same manner, but not quite so conveniently.

For ordinary I-beams the longest steam-railway spans are about twenty (20) feet when four lines of stringers per track are adopted, but by employing the thirty (30) inch special sections of the Bethlehem Steel Company the limit can be increased to about thirty (30) feet for fairly-heavy engine-loads. By using six lines of beams per track the limit can be increased to about thirty-five (35) feet, for which span the depth is only one-fourteenth (1/14) of the length, a ratio common enough in England, but ob-

jected to by most American railway engineers on account of the great deflection that it permits.

DECK PLATE-GIRDER SPANS

Although plate-girders are of necessity as unscientific structures as a bridge specialist ever has to design, they are without doubt the most satisfactory type of construction possible for short spans. Their superiority over articulated trusses is due to the following reasons:

First. Owing to their compactness they better resist shock and check vibration.

Second. They have fewer critical points where overstress is likely to exist because of faults of either designing or workmanship.

Third. A number of loose rivets lying close together will do far less harm in a plate-girder than in an open-webbed one.

Fourth. The cost of manufacture per pound of metal is a little less.

Fifth. Owing to the steady demand for plate-girder structures and the comparatively small number of the sections of metal used in their manufacture, it is easy to obtain quickly the materials required; and the work on the metal is of a simple character. For these reasons plate-girder spans can generally be purchased with less delay than open-webbed girders.

Sixth. The cost per pound for erection is decidedly less, excepting where the conditions are unusual, and the cost of painting is comparatively small.

Seventh. They can be overstressed without danger much higher than open-webbed girders.

Eighth. They are less liable to injury by accident than articulated trusses.

Ninth. They are more easily painted, and are more accessible to examination for rust.

Tenth. The cost of maintenance is less, owing to the absence of small parts and details that might work loose under traffic.

The ordinary limit of length of plate-girder spans is about one hundred (100) feet, but that limit has often been surpassed by twenty-five (25) or thirty (30) per cent for simple spans and by much more for swing spans. Usually it is the difficulty in shipping very long plate-girders from bridge shop to site that determines the superior limit of such spans. The loading of long girders on cars for shipment is quite an art, and it should be entrusted only to men experienced in such loadings; for, otherwise, the metal is liable to be injured in transit or the cars to break down, or some other trouble is likely to happen before they reach their destination. Some engineers believe that the liability to injury of long plate-girders in shop, transit, and field should limit their length to one hundred (100) feet; but the author is not of this opinion, for he thinks that by taking proper pre-

cautions the danger can be pretty nearly eliminated. About as long a plate-girder as has ever been shipped in one piece was one of one hundred and thirty-two (132) feet. It required four flat-cars to transport it. Longer plate-girder spans than this have been built, notably tubular bridges and swing spans, but they were shipped in parts and assembled at site. This expedient for simple spans is really permissible only in case of bridges to be sent to foreign countries, and it is to be avoided if possible even then, because it is sometimes difficult to obtain a satisfactory job of field-riveting when making the splices, although the use of pneumatic riveters tends to reduce materially the force of this objection.

As far as economics is concerned, it may be stated that, if deck plate-girders are feasible for any opening, they are more economical than truss spans up to a length that is prohibitory for shipment. As the depth of a very long plate-girder is generally from one-tenth (1/10) to one-twelfth (1/12) of the span, the requirements of underneath clearance often bar out deck plate-girders and necessitate either half-through plate-girders or through trusses.

Again, the great depth required for very long plate-girder spans often sets the limit for span-length because of shipping requirements. Some railroads have tunnels and overhead crossings which are lower than custom is now requiring; and very deep girders loaded on flat-cars might not be able to pass—nor could such girders be placed flat, because then the horizontal clearance would be encroached upon.

Half-Through Plate Girder-Spans

The economic limit of length for this type of structure is materially less than that of the type just treated, because of the necessity for using a steel floor. On this account it has not the advantage over the through-truss bridge which the deck-plate girder structure possesses. For a length of one hundred (100) feet the weight of metal in the latter type exceeds that in the former, by from five (5) to fifteen (15) per cent, the smaller figure being for the lightest live-loads and the larger for the heaviest. Of course, the cheaper metal of the plate-girder type would tend to offset its greater weight, but, in order to make the costs of the two 100-ft. steam-railway-bridge spans the same, the ratio of pound prices for metal erected in the girders and trusses themselves would have to be from 1.1 to 1.3—a condition of market that is unusual. But as, for various good reasons, it hardly seems advisable to build through, steam-railway spans shorter than one hundred (100) feet, it is well to adopt this length as the superior limit for half-through plate-girders and deck plate-girders in standard railway bridges. For electric-railway bridges and highway bridges, this limit might advantageously be reduced to about seventy-five (75) feet.

DECK, OPEN-WEBBED, RIVETED SPANS

Some engineers entertain the notion that for short deck-spans an economy can be effected by using open-webbed girders instead of plate-girders. If there be no objection to increasing the depth considerably, some metal can be saved in this way; but, if the same depth must be employed for both types, there is but little, if any, saving in weight, provided that the detailing be done properly—besides the pound price of the manufactured steel is a trifle greater for the open-webbed structure.

Some years ago there were designed for a transcontinental line a number of plate-lattice-girder spans. Their *raison d'être* was supposed to be primarily their ability to pass water through them when submerged, but secondarily, economy. The designer claimed that they effected a saving of metal amounting to about fifteen hundred (1500) pounds for an eighty (80) foot, single-track span, and that the pound price for their manufacture was no greater than that for ordinary plate-girder work. The author once used plate-lattice girders for the cross-girders of the Union Loop Elevated Railroad of Chicago, but his object was simply to evade a troublesome clause in the city ordinance. The webs of these cross-girders were solid near mid-span and at the ends, and were open near the quarter points, while those of the railroad girders previously mentioned were solid at the ends and open over more than the middle half of the total length. As far as the author's experience goes, it takes just as much metal to build the webs open, and the pound price for the finished metal is a trifle greater than it is for ordinary plate-girder construction. The fact that this same railroad, when drawing up a set of standard plans a few years later, discarded the plate-lattice girders is a pretty sure indication that the advantages claimed for them were more imaginary than real. It is true, of course, that in case of submergence they would pass a certain amount of water through their webs; but it is seldom that a railroad company will build a bridge of any kind so close to the high-water mark as to run any risk of its being submerged.

In respect to the economics of deck and through riveted spans, it may be stated as a general proposition that, although the former sometimes require more metal than the latter, they effect a great saving in the cost of the piers, and hence are to be adopted whenever permissible. Deck spans are cheaper *per se* when the ties can rest on the chords. This arrangement works well with double-track bridges having trusses spaced about twenty feet centers with two lines of stringers.

The question of the comparative economics of pin-connected and riveted spans is treated at length in the next chapter.

In respect to the comparative economics of the various kinds of trusses, it might be stated that a very few of them have stood the test of time, all freak and expensive styles having been discarded, the only types used

today being the Pratt, Petit, Triangular (both simple and sub-divided and including the Warren), and K trusses.

Comparing Pratt and Petit truss-spans, for which there is no difference worth mentioning in the pound prices of the metal, the weights per foot (and therefore the costs) are alike for single-track spans of three hundred (300) feet, and for double-track spans of three hundred and fifty (350) feet; but both constructive and æsthetic reasons generally necessitate limiting the lengths of Pratt trusses to about three hundred and twenty-five (325) feet.

In respect to the comparative economics of the Pratt and Triangular trusses, there seems to be a difference of opinion amongst bridge engineers. The author has found very little variation in their total weights of metal, with occasionally a slight economy in favor of the Triangular truss. That truss has the practical advantage that changes in chord stresses occur at only every other panel point. This often makes it possible to section the chords more economically.

As explained at length in Chapter XI, for continuous trusses of very long span, the Triangular truss has quite an advantage over the Petit truss.

The K truss is applicable for long spans only, and, therefore, is in competition with the Petit truss and not with the Pratt. Its principal claim for economy lies in ease and simplicity of erection, but it also has a tendency to reduce the high secondary stresses inherent in the Petit type. It was employed to advantage in the design of the great Quebec cantilever bridge.

The statements made in this chapter apply mainly to railway bridges and heavy highway structures. For light highway bridges some of them might require a slight modification.

CHAPTER X

COMPARATIVE ECONOMICS OF RIVETED AND PIN-CONNECTED BRIDGES

For at least a dozen years near the close of the last century there was waged in the technical press and orally when bridge engineers met (especially if there were both Europeans and Americans present) a war of words concerning the relative merits of riveted and pin-connected bridges; but all arguments that were advanced failed to solve the disputed question. Time and the steady development of the real science of bridge designing, however, gradually brought about changes of opinion among the leaders in that specialty; and the matter was finally settled upon a compromise basis.

The advocates of riveted structures used to claim greater rigidity and an increased chance for safe passage by a derailed train, while the endorsers of pin-connected construction used to rest their case mainly upon the theoretically-correct distribution of stresses by articulated joints and the smaller amount of metal needed for building. It is true that there was then a wide divergence in the weights of metal required for constructing riveted and pin-connected bridges to carry the same live loads; and for this there were two salient reasons. First, the riveted structures were of the lattice-girder type, having two, three, or even four systems of triangulation, thus involving much idle or superfluous metal in the main members of the web and even more in the numerous connecting plates and fillers; and, second, the pin-connected structures were proportioned essentially for the theoretical stress-requirements, irrespective of proper minimum sections, thus cutting the weight of metal down to an absolute minimum.

Gradually, though, these two types approached each other in weight, the lattice-trusses being abandoned for the riveted single-intersection types, such as the Warren and the Pratt, and experience in operation showing in pin-connected trusses the necessity for stiffening the abnormally-light members so as to increase the rigidity and check the vibration. Today good designers of riveted structures intersect all the axial lines of main members at panel points just as carefully as do the designers of pin-connected structures; and, hence, the prime objection to the former type, viz., its unscientific intersection of symmetry lines, vanishes *in toto*. It is true, though, that there remain the unavoidable secondary stresses, but these exist also to a small degree in pin-connected bridges because of the friction of the pins in their holes and the consequent failure of the joints to function as actual articulations. The employment of eye-bars certainly

cuts down the quantity of metal, because, in the members where they are used, the weight of steel for both main sections and details is an absolute minimum; and, in general, pins weigh less than connecting plates. Whether secondary stresses receive proper consideration or not, a pin-connected bridge is somewhat lighter than the corresponding riveted one, and therefore ought to be less expensive. It is true that the fine shopwork requisite for the proper manufacture of pins and for the drilling of pin-holes makes the pound price for fabrication greater in the pin-connected structure; but this is offset more or less by its lower pound cost for erection, owing to the smaller number of field rivets to be driven, the shorter time required to make safe against loss of span by washout of falsework, and the reduction in overhead expense effected by minimizing the time for field operations.

From a study of the diagrams of weights of metal per lineal foot of span for pin-connected and riveted-truss bridges given in Chapter LV of "Bridge Engineering," it is found that the weights of the latter exceed those of the former by the following percentages:

SIMPLE SPANS

Span Lengths	Percentage of Increase
200′	4
300′	5
400′	6
500′	8
600′	11
700′	14

CANTILEVER SPANS

Main Openings	Percentage of Increase
600′	6
1000′	9
1400′	13

It is seen from these tables that the percentage of saving by using pin-connections increases gradually with the span-length. This is because of the effect of the reduced dead-loads.

Notwithstanding the fact that pin-connected-truss bridges are somewhat lighter, and possibly somewhat cheaper, than the corresponding riveted-truss bridges, the latter are decidedly preferable for all short or medium-length spans of railway structures (both steam and electric); because in short, pin-connected spans the vibration from rapidly-passing loads is so great that the motion of the eyes on the pins causes such a grinding of both that eventually the structure has to be replaced on that account. This was shown long ago to be the case in pin-connected elevated railroads; and lately it has been found necessary to replace pins in a number of rail-

road bridges. The author saw an elevated railroad in Kansas City removed practically for this cause alone, the pins having been cut into and the eyes elongated as much as an eighth of an inch. In the author's opinion, a steam-railroad-bridge span should be pretty long before pin-connections are resorted to—say 500 feet for simple spans and 900 feet for the main openings of cantilevers; but for highway and electric-railway bridges these lengths may be cut down from twenty-five to thirty-five per cent. The point is one to be settled by individual judgment based upon experience—not prejudice. The size of the appropriation available for construction may quite legitimately be a ruling factor in making the decision, because a pin-connected highway span of five hundred, or even four hundred feet, does not make a bad bridge; although, if the traffic be great, such a structure certainly is inferior to a riveted one, in that it will vibrate more and will possibly be shorter lived. But if the live load assumed for the designing be never greatly exceeded, and if the structure be always kept properly painted, it would probably require more than a century of use to wear the pins and pin holes to any dangerous extent.

The discovery of a high-alloy of steel that can be manufactured at reasonable cost, and the development of a satisfactory and absolutely-reliable method of heat treatment thereof for eye-bars may bring the pin-connected structure once more into vogue; but it will be for long spans only, and preferably for highway structures.

CHAPTER XI

COMPARATIVE ECONOMICS OF CONTINUOUS AND NON-CONTINUOUS TRUSSES

THIS chapter is mainly a reproduction of a joint paper by Mr. H. Malcolm Priest and the author, lately presented to the Engineers' Society of Western Pennsylvania.

For many years past bridge engineers have held differing opinions concerning the advantages of continuous trusses as compared with the corresponding non-continuous ones. Some claimed a great saving in weight of metal from continuity while others felt sure there was none. The author had been under the impression that the advantage claimed for the continuous truss was mainly due to the ignoring of the effect of reversing stresses; and, as will be seen later, in this opinion he was partly right and partly wrong.

Under certain conditions it is not bad practice to use continuous trusses, and under others it is, irrespective of the question of economics. When the foundations of the piers are solid rock or other very hard material, continuity is permissible; but when they are piles or comparatively soft material without piles, it is better to forego any possible saving of metal rather than to run the risk of unequal settlement of piers and the consequent upsetting of stress distribution throughout the trusses from end to end of structure.

The most notable example of continuous trusses in America, or, as far as the author knows, anywhere else in the world, is the Sciotoville Bridge over the Ohio River on the line of the Chesapeake and Ohio Northern Railroad. At this location continuous trusses were permissible, for the reason that the pier foundations are solid rock at no great distance below the bed of the stream. That structure consists of two continuous, double-track-railway spans of 775 feet each. It was designed and engineered by Dr. Gustav Lindenthal, Consulting Engineer, and was completed in 1917.

Desiring to have this comparison of weights of metal for continuous trusses conform as closely as possible with actual conditions, the author assumed the outlines of the Sciotoville structure, and contrasted it with a bridge of two simple spans, each made five (5) feet shorter, so as to allow for the distance between centers of pedestals on the middle pier, using practically the same panel-lengths as those of the Sciotoville Bridge and economic truss-depths for the simple spans based upon the information given in "Bridge Engineering."

These two layouts are shown in Figs. 11*a* and 11*b*.

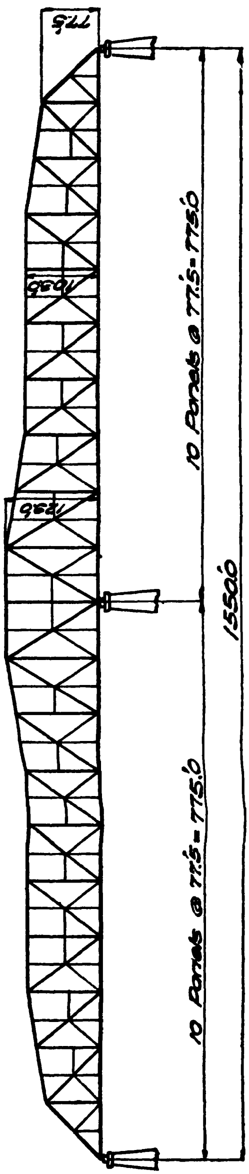

Fig. 11*a*. Layout of Petit-Truss, Continuous Spans.

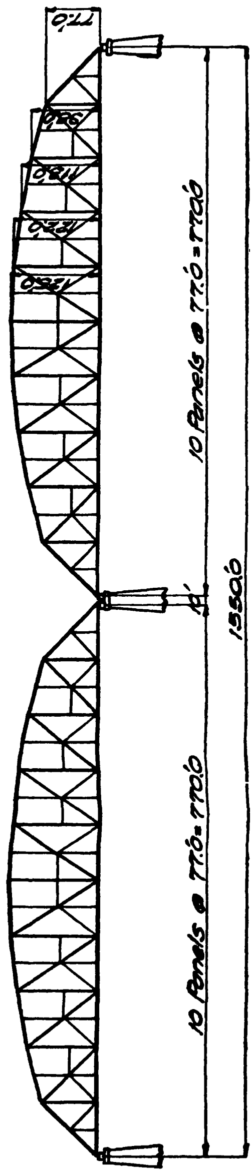

Fig. 11*b*. Layout of Petit-Truss, Non-Continuous Spans.

The first series of computations was made upon the basis of adopting the Petit-truss type; and there was assumed a unit load of 1,000,000 lbs. applied at one panel point at a time, the stresses therefrom being found for each main member of each span affected by the loading. These

stresses were summed up for greatest tension and greatest compression on each piece, in order to determine by slide rule the live load stresses. The live load assumed for each track was the author's Class 60 loading.

In order to save time and labor, a constant percentage for impact was included in the live load itself instead of varying the percentage amounts to be added to the live-load stresses in the different web members. This approximation, of course, caused certain errors in web stresses; but their effects on the two contrasted types of structure were practically alike, and, therefore, did not affect the correctness of the comparison. The reactions for concentrated loads in the continuous-truss structure were obtained by the Theorem of Three Moments. No attempt was made to correct later the stresses thus found by the more exact method of least work; for the reactions obtained in that manner by the designers of the Sciotoville Bridge indicated that the difference in total weight of metal caused thereby was trifling.

The finding of the live-load stresses was a comparatively simple matter, but the determining of the dead-load stresses was much more arduous, because sometimes the correct distribution of the metal between the various panel-points was not ascertained until the third trial. No attention was paid to wind stresses; because, in double-track railway-bridges of long span and heavy live-loading, the excess intensities of working stresses allowed in modern bridge specifications for combinations of wind stresses and other stresses result in rendering wind stresses in the trusses entirely negligible.

After the live-load stresses and the dead-load stresses for both the continuous and the non-continuous spans had been computed, they were combined, and the maximum stress on each piece for both tension and compression was recorded. Then the sectional areas were determined by the specifications of Chapter LXXVIII of "Bridge Engineering," ignoring, however, all effects of reversion; after which the total weights of metal in main members were figured for both layouts, and to them were added the proper percentages to cover weights of details, thus giving the comparing weights of metal for the two types of structure under consideration. Much to the author's surprise, the weights thus found were so nearly alike that their difference amounted to a small portion of one per cent—so small, in fact, as to be negligible.

It had been intended to make an entirely new set of sectional areas and compute the resulting weights of metal for both types on the basis of caring for reversing stresses in accordance with the method provided in the before-mentioned specifications; but this was found to be unnecessary, because members in which reversion occurred were very few, and both the direct and the indirect effects thereof were readily determined. By "indirect" effect is meant in this case the increase in weight of metal due to augmentation of dead load caused by provision for reversal. Here again was a surprise, for the effects on the two types were exactly alike. These

computations showed that, for long-span, double-track, steam-railway bridges of the Petit-truss type, there is no economy of metal whatsoever in making adjacent spans continuous over the piers, and that the matter of caring for or ignoring the effects of reversing stresses does not in any way influence the economics.

These results were so decided and the coincidence of weights was so exact that at first the author thought the entire question was settled; but it was suggested by Mr. Shortridge Hardesty, his principal designing engineer, that, if the divided-triangular truss adopted for the Sciotoville Bridge were investigated, a different result might be found; and it was decided to make the test. The layouts of trusses for the new comparison are shown in Figs. 11*c* and 11*d*. The computations were all prepared exactly as before, and it was found necessary to make two sets of figures for the continuous-truss layout before the correct dead load was determined. The findings of this second set of computations were as follows:

In the simple-truss spans the weights of metal for the divided-triangular and the Petit types were nearly alike, the slight difference which there was being in favor of the former; and the continuous-truss type showed a gain of twelve per cent over the non-continuous type when reversion was ignored and eleven per cent when it was properly provided for.

The computations made up to this stage of the investigation settled the economics for long-span, steam-railway bridges; but it was seen by *a priori* reasoning that the results would probably be somewhat different for standard highway bridges, consequently it was decided to repeat the calculations for the latter structures. To this end the total loadings (i.e., live load plus dead load) were retained, but a different division thereof was made by diminishing the live loads and increasing the dead loads, so as to correspond, as nearly as may be, with the ratios of those loads in modern highway bridges having a paved roadway, reinforced-concrete base-slab, and reinforced-granitoid sidewalks for the span-length under consideration.

The result of this third set of computations showed that for the divided-triangular-truss layout there was an economy of twenty-two per cent in favor of the continuous trusses, and that reversing stresses affected very few members and those so slightly that the influence of reversion could be completely ignored. It was not thought worth while to repeat these highway-bridge calculations for the Petit-truss layouts.

The computations made thus far settled the comparative economics of continuous and non-continuous trusses for long-span bridges, both railway and highway, also those of the Petit and the divided-triangular trusses for such structures; but it could not properly be assumed that what was found to be true for long spans would apply also to short ones, consequently it was decided to make a new set of computations for comparatively-short-span layouts for steam-railway bridges of both the divided-triangular truss and the Pratt-truss types.

By reducing all the truss dimensions to one half, it was practicable to employ without change the previously-determined index stresses (those

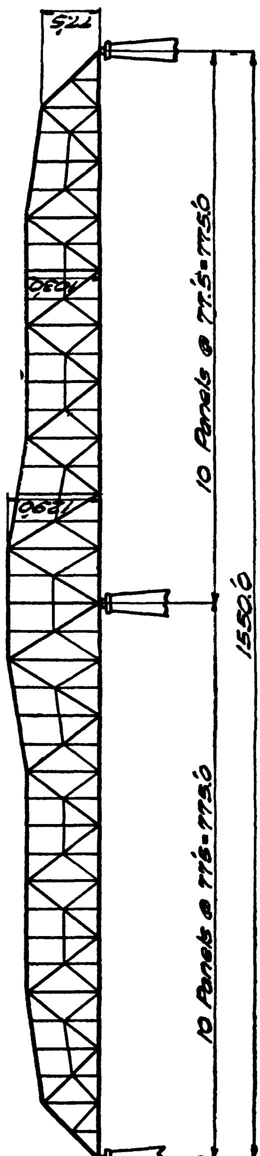

FIG. 11c. Layout of Subdivided-Triangular-Truss, Continuous Spans.

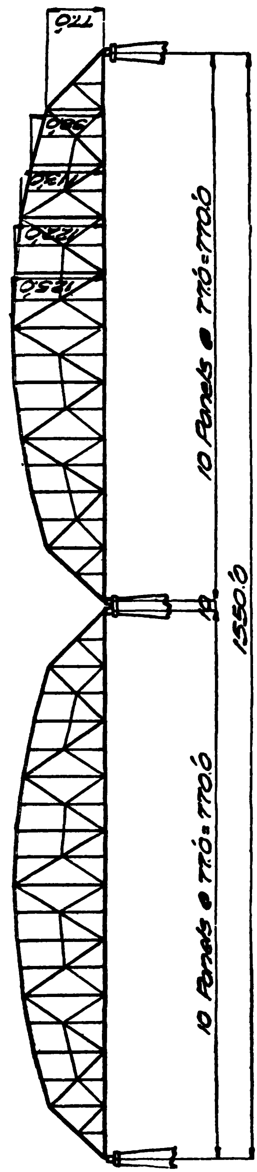

FIG. 11*d*. Layout of Subdivided-Triangular-Truss, Non-Continuous Spans.

found from assumed unit loadings), and thus the labor of figuring was greatly reduced. As before, the dead-load stress-computations were repeated until the assumed and the resulting dead loads at the different

panel points were in close agreement. The results of this fourth set of calculations were as follows:

The divided-triangular-truss figures indicated a gain of seven per cent for the continuous-truss layout over the non-continuous one when reversing stresses were ignored and *no gain at all but simply a stand-off* when they were properly provided for. But, when the Pratt-truss was employed, the non-continuous-truss layout showed a gain of two per cent over the continuous-truss one when reversing stresses were ignored and five per cent when they were properly provided for.

The said results also indicated for short, simple-truss spans that, when reversals are ignored, there is no difference in weight of metal between steam-railroad bridges of the Pratt-truss and the divided-triangular-truss types; but that, when reversals are properly provided for, the latter has an advantage of five per cent.

It was not deemed worth while to compute the economics for highway bridges of short spans; but it might be inferred by *a priori* reasoning, based on the preceding results, that, in the case of the divided-triangular trussing, the continuous spans would have an advantage of thirteen per cent when reversals are ignored and twelve per cent when they are properly cared for—also that in the case of the Pratt trussing there would be no material advantage in continuity.

Without making another set of computations, the author would not care to employ *a priori* reasoning for the determination of the comparative economics of continuous and non-continuous trusses in long-span, highway bridges of the Petit-truss type, excepting that it seems pretty safe to assume that the Petit trussing would have no advantage over the divided-triangular trussing, and that the continuous trusses would probably show a small advantage over the non-continuous ones.

All the actual results of the calculations made for this study are collected in the two following tables, and are expressed in ratios, unity standing for weights of continuous divided-triangular trusses when the effect of reversals is ignored.

TABLE 11*a*

SUMMARY OF WEIGHT RATIOS

Divided-Triangular Trussing

Span Length in Feet	Type of Bridge	Reversals Ignored		Reversals Considered	
		Continuous	Simple	Continuous	Simple
775	Railway	1.00	1 12	1 03	1 14
775	Highway	1 00	1 22	1 00	1 22
387½	Railway	1 00	1.07	1 10	1 10

TABLE 11*b*

PETIT OR PRATT TRUSSING

Span Length in Feet	Type of Bridge	Reversals Ignored		Reversals Considered	
		Continuous	Simple	Continuous	Simple
775	Railway	1 13	1 13	1 16	1 16
387½	Railway	1 09	1 07	1 20	1 15

In determining the comparative economics of continuous and non-continuous trusses for any proposed bridge, the application of the preceding findings would have to be somewhat modified in case the structure has to be erected by semi-cantilevering. Under such a condition the continuous trusses have an advantage over the non-continuous ones, at least to the extent of the extra metal required by the toggles for the latter over the center pier. Again, it is probable that some of the lighter truss members in either type will need reinforcing for erection stresses; and this consideration is likely to affect the non-continuous trusses more adversely than it does the continuous ones.

SUMMARY OF CONCLUSIONS

Summarizing the results of the entire investigation, the following conclusions are reached:

First. For long spans the divided-triangular trussing is decidedly superior to the Petit trussing for bridges with continuous-truss spans, but not much so, if at all, for those of simple-truss spans.

Second. For long spans there is an important saving of metal by the adoption of continuous trusses, and the said saving is nearly twice as great for standard highway bridges as for modern, double-track railway-bridges.

Third. For long-span bridges the method of treating the matter of stress reversal has practically no effect upon the comparative economics of continuous and non-continuous trusses.

Fourth. For comparatively-short-span, steam-railway bridges, the continuous truss has a small advantage over the simple truss only when the divided-triangular trussing is used and stress reversals are ignored. In all other cases the comparison is either a stand-off or in favor of the simple truss.

Fifth. For comparatively-short-span, steam-railway bridges, the divided-triangular trussing is generally more economic of metal than the Pratt trussing.

Sixth. In no case should either the Pratt or the Petit truss be employed for continuous spans, because in these the divided-triangular truss is more economic.

Seventh. In case of a structure requiring erection by semi-cantilevering the continuous truss will possess an advantage over the non-continuous one, at least to the extent of the weight of extra metal required by the toggle for connecting temporarily over the center pier the inner hips of the two simple-truss spans. This advantage may even extend to the reinforcement for erection stresses, but if it does, the gain thus involved can never be very great.

CHAPTER XII

COMPARATIVE ECONOMICS OF SIMPLE-TRUSS AND CANTILEVER BRIDGES

About the time that cantilevers came into vogue, some twenty-five or thirty years ago, certain bridge designers entertained a wild idea to the effect that the new type involved some special virtue or feature of excellence or else that it was economic in first cost; because many cantilever bridges were built in places where simple-span structures would have been far better and cheaper. Possibly the thought of establishing an innovation induced some of the designers of those bridges to prefer the cantilever type to that of the simple truss. What a pity it is that such designers did not devote their time and energy to an attempt to introduce the steel-arch bridge into American practice! Had they done so, probably they would have been successful; because there is often true economy in the arch—besides it is far more aesthetic than either the cantilever or the simple truss. A long-span, cantilever bridge can be made agreeable to the eye by using artistic outlines and a well-studied web-system; and, again, its simple vastness produces a pleasing impression upon the beholder; but a small-span cantilever is ugly and causes a trained intelligence to propound to itself the question "why and wherefore?" without receiving a satisfying answer.

It is true that for certain fairly-narrow crossings the water is so deep, or the current is so swift, that the use of falsework is out of the question, and that the adoption of cantilevering during erection is necessitated. Such conditions, however, do not require cantilever bridges, but semi-cantilevers, i.e., structures that are cantilevers during erection and either simple spans or arches afterwards. This method of erection for simple-truss spans was first evolved by the author some twenty-five or thirty years ago, but was not actually used by him in construction until some time later. A description of it will be found in Chapter XXV of "Bridge Engineering."

It is evident to any engineer who gives the subject due consideration that a cantilever bridge is less rigid than the corresponding simple-truss structure, because its vertical deflections under live load are necessarily larger, thus permitting more vibration as well as greater irregularity in the track grade; hence, for steam-railway bridges, other things being equal, the simple-truss layout should be chosen—or even if it should cost somewhat more, because rigidity is an important consideration in the operation of steam-railway trains. For highway and electric-railway bridges, though, it is not of such great importance; consequently, if in these structures the

cantilever should show even a small economy in the comparison, it would be well to adopt it.

The question of what is the economic limit of length of simple-truss spans as compared with cantilevers is still a mooted one. Professors Merriman and Jacoby, on page 119 of Part IV of their excellent treatise on "Roofs and Bridges," state that the economic limit for simple spans was probably nearly reached in the building of the five hundred and eighty-six (586) foot span over the Great Miami River at Elizabethtown near Cincinnati; but the author has had occasion to compare simple-truss spans of somewhat greater length than that with the corresponding cantilever structures and has found them more economic. The continuity of cantilever spans in resisting wind loads lowers the requirement for minimum width from one-twentieth (1/20) to one-twenty-fifth (1/25) of the greatest span-length, and hence, because of substructure considerations, gives an advantage to the cantilever type that in certain extreme cases will more than offset its disadvantage of greater weight of truss metal.

This question of when to pass from simple-truss spans to cantilevers is not affected very much today by the last consideration, because bridges with spans long enough to necessitate the comparison are often so wide as to cause it to be ignored. For instance, one seldom hears any more of a single-track railway bridge having a span longer than four hundred (400) feet; and first-class, double-track, steam-railway bridges have a clearance of twenty-eight (28) or, preferably, thirty (30) feet, thus making the distance between central planes of trusses from thirty-two (32) to thirty-four (34) feet. The limiting simple-truss span-length established by good American practice for the latter dimension is six hundred and eighty (680) feet, and for cantilevers it is eight hundred and fifty (850) feet.

Of still greater importance are the special requirements that govern the layout at each site. Fig. 12*a* (which is a reproduction of Fig. 55*aaa* on page 1271 of "Bridge Engineering"), shows typical layouts for cantilever bridges. There is still another type, consisting of equal (or nearly equal) spans with short cantilever arms, that is discussed later in this chapter.

The Type-C-cantilever bridge, which has three spans of practically equal lengths, will first be considered. It will be compared with a corresponding structure having three simple-truss spans. These layouts apply where the distance between end piers is fixed, while the intermediate piers can be placed where desired. Fig. 12*b* gives the comparing weights for pin-connected, double-track-railway bridges. From its curves one can see that the span of equal cost is about six hundred and thirty (630) feet. It may be possible to reduce the cantilever weights by varying the sizes of the openings and the relative length of suspended span to cantilever arm; but, even with such changes, it is not likely that the span-length for equal weights of metal would be as low as six hundred (600) feet.

In the case of highway bridges, the weights of metal per lineal foot in

cantilevers would be about the same as those for simple-truss spans of five hundred and fifty (550) or five hundred and seventy-five (575) feet.

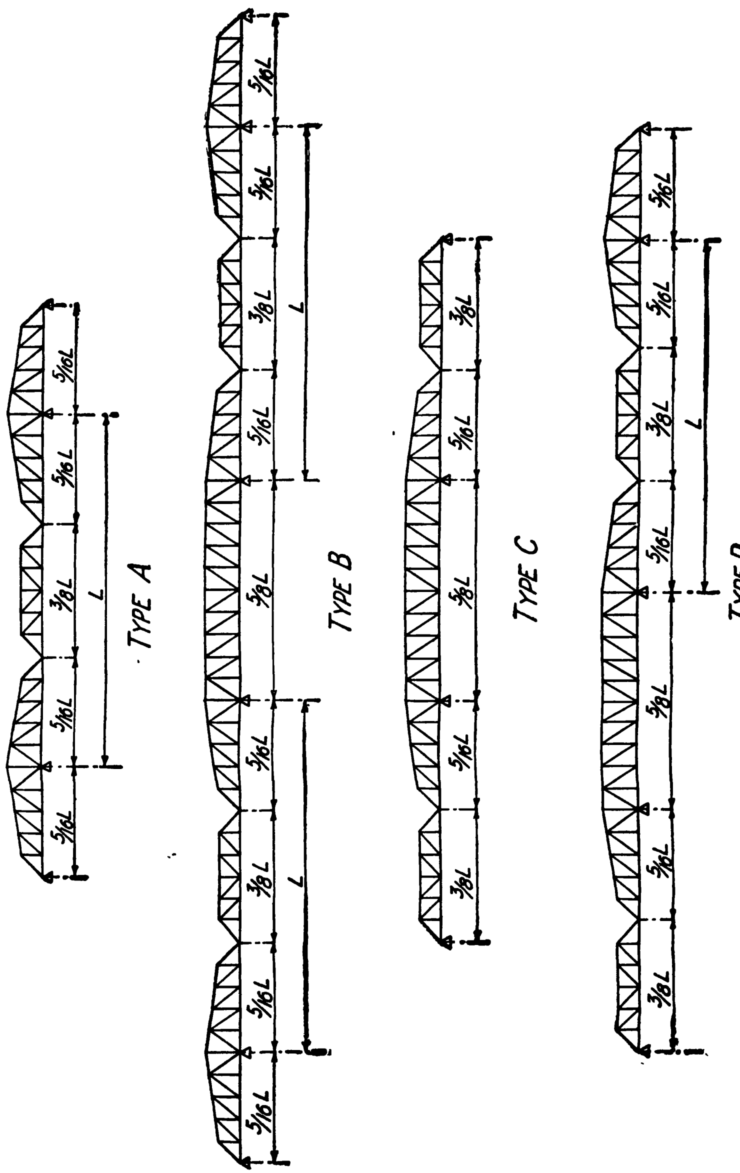

Fig. 12a. Typical Layouts for Double-Track-Railway, Cantilever Bridges.

It is unnecessary to consider the comparative economics of simple-spans and cantilevers for electric-railway bridges pure and simple. Nobody

is ever going to build any of them of long-enough span to bring up the question, because such expensive structures would surely be constructed to accommodate highway traffic also.

The type of cantilever under discussion (Type C of Fig. 12*a*) is not well adapted to crossings where falsework cannot be employed; and the use of the simple-span layout, in which the central span can be cantilevered out from the side ones, is frequently preferable on this account. Also the adoption of the three duplicate spans or even the duplicate side spans will

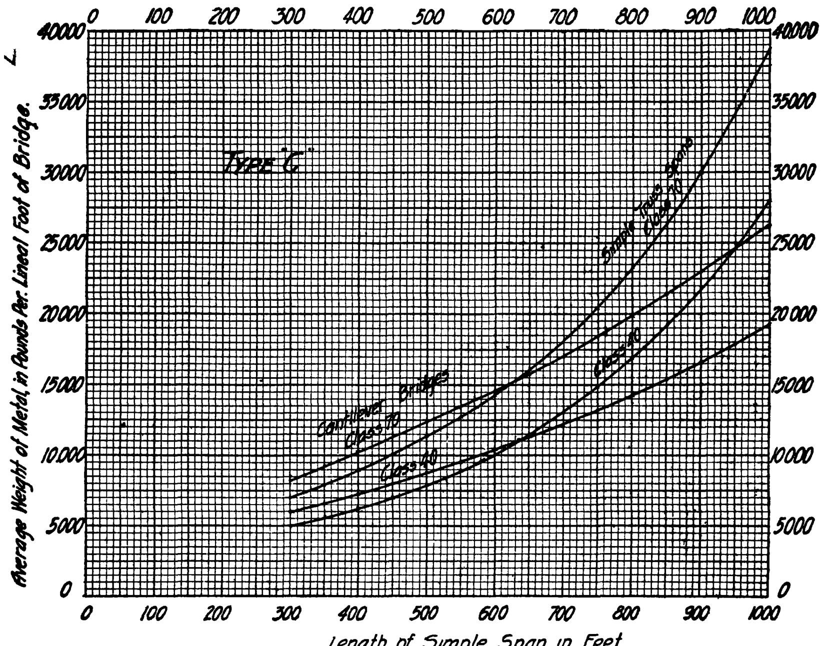

FIG. 12*b*. Comparative Weights of Metal for Double-Track, Simple-Truss Bridges and Type-C-Cantilever Bridges.

usually cause a small reduction in the pound price of the manufactured metal, as compared with that for the cantilever.

The cantilever layout denominated Type A in Fig. 12*a* is the most common of the four types illustrated. It is generally used for a long-span structure where the length of the central openings is fixed, while the lengths of the end spans can be made much shorter. Also, it is more likely to be adopted where the central span cannot be erected on falsework. For these reasons it cannot fairly be contrasted with a simple-truss, equal-span layout, nor with the Type-C cantilever. It may very properly, though, be pitted against a simple-truss layout with unequal spans—in fact the same spans as the cantilever layout has. Fig. 12*c* gives curves of weights for such layouts, for pin-connected, double-track, steam-railway

bridges. It will be noted that the span-length for equal weights is about 620 feet. If the central span must be erected by cantilevering, extra metal will be required for the simple-truss layout, amounting to about 8 per cent of the truss-weight. The span-length for equal weights will then be a little over 500 feet.

If the side-spans in the simple-truss layout be replaced by steel trestle, the span length for equality of weights will be about 700 feet when the central span is erected on falsework. This layout, evidently, cannot be used when the central span must be cantilevered out.

The Type-A cantilever of the proportions shown in Fig. 12a is usually an uneconomic layout to adopt when the distance between the end piers is a

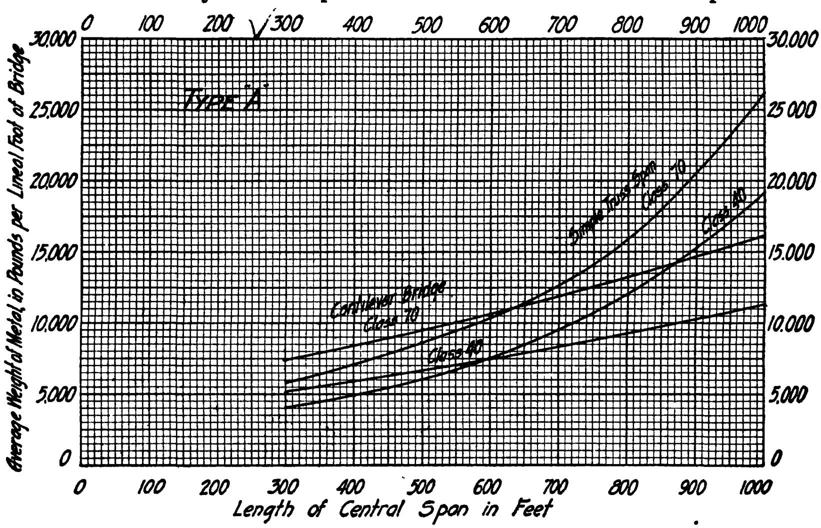

FIG. 12c. Comparative Weights of Metal for Double-Track, Simple-Truss Bridges and Type-A-Cantilever Bridges.

fixed quantity, while the two intermediate piers can be placed where desired. For such a location there should generally be used three simple-truss spans, a Type-C cantilever, or a cantilever with the end spans nearly as long as the central span. As before stated, this latter layout is discussed subsequently in this chapter. In many cases, however, the piers of the Type-A cantilever with a long central span will be much cheaper than those of the simple-span bridge, on account of their being nearer to the banks of the river. For such a crossing, the total cost of the simple-span structure can sometimes be reduced by lengthening the center span and shortening the side-spans; and the most economic layout for the simple-span bridge should first be found, and its total cost then compared with that of the cantilever structure.

There is an economic consideration of some importance in the comparison of simple-truss and cantilever bridges which, as far as the author knows,

has never yet been given any attention—at least none in print. It indicates within rather narrow limits a slight economy for the cantilever type, but the amount thereof and the location of the said limits are dependent upon several considerations, among which the most important are the following:

A. Average length of spans considered.
B. Ratio of live load plus impact to total load.
C. Method adopted for combining reversing stresses, when proportioning sections of members.

The best conception of this matter of economics can be obtained from a dissertation based upon an assumed layout of simple-truss spans, all of equal length,—for instance, a long succession of like spans of two hundred feet each, the panel-lengths being twenty feet. If now we extend the trusses of every other span one panel beyond each of its piers and suspend from the cantilevered ends thus formed the shortened intermediate spans, we shall have a cantilever bridge that will effect a saving in weight of metal in every span. It is evident without any figuring at all that the spans which contain the suspended trusses will weigh less than the simple-truss spans, because the suspended portion is decidedly lighter and the cantilever arms, being so short, cannot be very heavy. Again, the dead load stresses in the chords of the other spans are somewhat reduced, but probably not enough to permit of any reversion of stress when the span is empty with the adjoining spans loaded. The total stresses in chords are, therefore, materially smaller than those for the simple-truss spans, resulting in an economy of metal, notwithstanding the fact that there are two extra panel-lengths of top chord involved by the change from simple-span type to cantilever type, and that the vertical posts over the piers combined with the end main diagonals are somewhat heavier than the inclined end posts of the simple-truss span. On the other hand, though, the cantilever structure, having only one pair of pedestals per pier, involves a slight economy of "metal on piers" and permits the width of pier-top to be reduced a little below that required for the two pedestals of consecutive trusses in the simple-truss layout, which saving in some cases will extend from coping to bottom of caisson.

Next, let us assume that there are cantilever extensions of two panel lengths instead of one. There may or may not be a material saving of metal in those spans containing the suspended trusses in comparison with simple-truss spans, although there will be some reduction; for, while there is a decided lessening in the weight of metal per lineal foot in the suspended portion, the cantilever arms tend to become heavy. In the chords of the other spans, the dead-load stresses are made very small, permitting some reversion therein from the live loads on the adjoining spans; and the result will probably involve an increase in weight of truss metal. The net effect of the change in layout upon the structure as a whole is uncertain; but it is probably slightly uneconomic of metal.

If the assumption be made that three panel-lengths be cantilevered, it is almost certain that the total weight of metal in the structure will be augmented.

It is evident that the proportionate effect of the cantilevering under consideration is dependent upon whether the bridge is a railroad structure or a highway one with a paved roadway supported on a reinforced-concrete base, because the relative effect of reversion is far greater in the former case than in the latter; hence an amount of cantilevering of this kind that would be uneconomic in a railroad bridge might be truly economic in the corresponding highway structure.

As the span-length increases, the ratio of live load to total load decreases, and hence the proportionate effect on weight of metal due to reversing stresses diminishes. For this reason one can anticipate that the longer the average span the greater will be the relative importance of the economic feature of design under consideration.

Finally, the saving in metal (or the reverse) by this method of cantilevering is fundamentally dependent upon the manner in which the designing specifications take care of reversing stresses. If these be entirely ignored, as some engineers advocate doing, the cantilevering will effect a large economy of metal, even when the cantilever arms are comparatively long; whereas, if these stresses of opposite sign are cared for by adding to the larger three-quarters of the smaller and proportioning for the sum, the saving will be but little, if any. The most approved and up-to-date practice is to add to the larger stress only one-half of the smaller; and in that event some economy may be anticipated, provided that the length of the cantilever arms be not too great.

If, with a layout such as is being considered, there be found for openings of equal size an economy in a certain amount of cantilevering, the question arises "would there not be a further saving of metal, if the lengths of the continuous spans were to remain fixed and those of the other spans were to be moderately increased?" The answer to this question, in all probability, is affirmative, although the economy involved would not be important.

In view of the preceding dissertation, it is evident that it is entirely impracticable to give any quantitative solution to this economic question, but that it must be solved for each case as it arises. In railroad bridge designing the matter is not important; because it is highly improbable that the value of the metal saved would offset the disadvantage of the reduction in rigidity that is unavoidable when changing from the fixed-span type to that of the cantilever. But in the case of a highway bridge with a heavy floor, it is an altogether different matter, because, as previously pointed out, rigidity is not so fundamentally important in highway structures as it is in railway bridges; and, moreover, the stiff concrete slab itself increases so greatly the rigidity of the steel construction that the detrimental looseness caused by the hinged attachments of the suspended span loses most of its importance.

CHAPTER XIII

COMPARATIVE ECONOMICS OF CANTILEVER AND SUSPENSION BRIDGES

THIS chapter is essentially a reproduction of a paper delivered by the author to the Western Society of Engineers at Chicago on September 15th, 1919. It is reproduced here practically in full for two reasons:

First. Outside of the membership of that society, the paper has been read very little, and it did not receive any written discussion; consequently, its contents, as far as this treatise and the engineering profession in general are concerned, are practically new material.

Second. Unless it be shown herein that most of the information which had been published about the subject prior to September, 1919, was wrong, there would not be much use in the author's stating that such is the fact and claiming correctness for the data thereon which he presents; because in engineering, as in all other walks of life, any man's word is as good as another's on a disputed point until one has given absolute proof of the correctness of his claim. Moreover, from the strictly-professional point of view, the demonstration of the author's findings and the record of the various steps which he took in his investigation ought to prove fairly interesting reading—at least to structural engineers and students of the specialty of bridge design and construction. But if any reader should feel averse to wading through this long chapter, he can easily arrive at the results of the somewhat-elaborate study by skipping to near the end of it, where he will find a résumé of conclusions.

The calculations for the suspension bridges were prepared upon the basis that the stiffening trusses were free at their ends; but later some more computations were made in order to determine the effect upon the economic deductions, under the assumption that the said ends were anchored, but not fixed, to the masonry. The results showed for both the longer and the shorter spans a decrease of nearly one hundred feet in the span-length for equal cost. This is not a serious difference, nevertheless it is well to remember that it exists.

The following is a reproduction of the previously-mentioned paper:

THE COMPARATIVE ECONOMICS OF CANTILEVER AND SUSPENSION BRIDGES

Under the title "Suspension Bridges and Cantilevers—Their Economic Proportions and Limiting Spans," Dr. D. B. Steinman in 1911 issued a little

book in the Van Nostrand Science Series; and in 1913 he produced a second edition of it with a few revisions and the addition of four folding plates.

In that treatise he draws the conclusion that "the critical span at which the suspension bridge becomes economically superior to the cantilever bridge is 1,670 feet." His calculations were made for a structure carrying four steam railway tracks between trusses and two exterior sidewalks on the lower deck, and a roadway with electric railway tracks between trusses on the upper deck, the total live load for the trusses being 18,000 pounds per linear foot, of which 12,000 pounds were for the steam railways. His profile shows bare bed rock, which, under the approaches, is approximately horizontal and a few feet above extreme high-water level. He figured his cantilever structures for main openings of 1,000 feet, 1,500 feet, and 2,000 feet, and his suspension bridges for main openings of 1,500 feet, 2,250 feet, and 3,000 feet.

While recognizing the value of Dr. Steinman's work and giving him due credit for his laudable energy and ambition, the author doubted the correctness of the main conclusion just mentioned, and in "Bridge Engineering" he wrote concerning it as follows:

"In order to evolve a mathematical demonstration of the problem, he (Dr. Steinman) had to make numerous assumptions more or less approximately correct. Without checking all of his mathematical work, it is evident that the professor has made as fair a comparison as he could, but his assumptions were so numerous and approximate that his conclusions must be taken with a liberal allowance for variation. . . .

"All these facts affect materially the question at issue, and it is probable that, if the changes implied were incorporated, the span length for equal cost found by the investigator would be considerably greater."

For a number of years the author has had the desire to settle this economic question; but the amount of labor involved had always appeared appalling. In truth, it was so, because Dr. Steinman spent most of his spare time for two years in making the computations for his investigation.

It is true that the author could easily have figured the weights of metal and the costs thereof for cantilever bridges by employing the diagrams which he prepared for his papers on "Nickel Steel for Bridges,"* and "The Possibilities in Bridge Construction by the Use of High-Alloy Steels,"* most of which diagrams were published in these papers; but not until after he had written Chapter XXVII of "Bridge Engineering" did he possess any quick method of computing the weights of metal and the costs of suspension bridges. In that chapter are presented for the first time a number of formulæ, from which, in conjunction with the numerous diagrams in Chapter LV of the same treatise, can be found quite readily the approximate weights of metal for all portions of suspension bridges.

In April, 1918, for the first time since the issuing of his book, the author found leisure to make the contemplated economic investigations. They

* Published in the Transactions of the American Society of Civil Engineers.

occupied all of his spare time for a month and a half, representing altogether some 300 hours of steady figuring. As in the case of his paper on "The Possibilities in Bridge Construction by the Use of High-Alloy Steels," he did all of the computation work entirely unaided, checking the results himself, but relying for their correctness mainly upon the regularity of the platted curves.

As his data on weights of metal in cantilever bridges were primarily for double-track-railway structures, his first investigation was made for that class of bridges, using the live loads, impact, and specifications indicated in the two previously-mentioned papers. For convenience of comparison, he assumed Dr. Steinman's unit prices for metal in place, but for substructure estimating he adopted the method which he has employed for many years, viz., using a unit price for concrete above low water, another for the mass of the pneumatic caissons with their superimposed cribs below low water, another for the corresponding mass below the same in box cribs filled with concrete resting on piles, and a price per lineal foot for those portions of the said piles projecting below the bases of the cribs. These unit prices are as follows:

Shafts and walls....	$15.00 per cu. yd.
Mass of pneumatic caissons with their cribs	25.00 per cu. yd.
Mass of box cribs, including enclosed portions of piles........	20 00 per cu. yd.
Piles projecting below base of crib.......	1.50 per lin. ft.

The unit prices for metal in place were as follows:

Wire cables	12.5¢ per lb.
Nickel steel............................	8.0¢ per lb.
Carbon steel in spans.....	5 6¢ per lb.
Carbon steel in trestle approaches....	5.0¢ per lb.

The costs of the railway tracks, the roadway pavements with their reinforced-concrete bases, and the reinforced-concrete sidewalks have been ignored when computing the total costs of structures, because they are common to the two classes of bridges compared.

In making the computations for this investigation, the author took the liberty of adopting several short cuts, such as assuming squared instead of rounded ends for all piers, using generally the method of "end areas" instead of that of the "prismoidal formula" when calculating volumes of masonry, carrying out quantities of materials and total costs to rather large limiting units, and estimating costs of certain parts by proportion from the previously-computed costs of similar parts of other structures. All these and many other short cuts for avoiding labor are perfectly legitimate when making comparative estimates, provided that they affect alike the compared types of construction, as they do in this case.

In the plotted curves of the accompanying diagrams no curve was

drawn through less than three located points, and in many cases the number was four or more. The regularity of all the curves proves that there was no error of any magnitude in the figuring which located the points thereof. It does not mean, however, that the author's calculations contained no errors. Unfortunately, several mistakes crept into the work, but the plotting invariably pointed them out and led quickly to their satisfactory correction.

In the diagrams the abscissæ represent main-span lengths in feet, and the ordinates show the total costs of structures in dollars, the recorded units being millions.

In figuring the weights of stiffening trusses for suspension bridges, the author made an important modification in one of the formulæ given in Chapter XXVII of "Bridge Engineering."* Equation 15 thereof has been employed without change, when wind stresses are ignored; but the following formula *for the weight per foot of both trusses* has been added to cover the case where the effect of the wind load is considered:

$$T=2.8\left\{3.9\frac{M_m}{ds}+\frac{3.26V_m}{s}\left(\frac{p^2+2d^2}{dp}\right)\right\}+\frac{8PL^2}{11bs}.$$

The corresponding equation when the wind stresses are ignored is:

$$T=2.8\left\{5.06\frac{M_m}{ds}+\frac{3.26V_m}{s}\left(\frac{p^2+2d^2}{dp}\right)\right\}.$$

The greater of the two values of T given by these equations is, of course, the one to use in the estimate of total weight of metal.

The division of total metal weight between carbon steel and nickel steel was made by the author's judgment, based upon the curves in his two before-mentioned papers and upon the assumptions of material distribution adopted when preparing the suspension-bridge computations. No error of any magnitude exists because of this assumed distribution, although, of course, the method employed is only approximately correct.

Whenever a proper weight curve for cantilever structures was not available, the author fell back upon the general curves for weights of metal in trusses and laterals that record the various double-panel weights in cantilever arms and anchor arms as multiples of the corresponding double-panel weight of the suspended span, which general curves were first given on Plate X of "De Pontibus," and afterwards were reproduced in Fig. 25j of "Bridge Engineering."

In establishing the general assumptions for the layouts of both cantilever and suspension bridges, with one exception they were made as favorable as possible for each type, that exception being that, for the sake of appearance, the anchor arms of each cantilever structure were made of the

* This modification has lately been incorporated in the third thousand of that treatise, now on sale.

same length as that of the cantilever arms, viz.: 0.3125 of the main opening, instead of the more economic value of 0.2 thereof. Concerning the correctness of the last claim for economy there is some dispute in the profession; but of this matter, more anon.

In the suspension-bridge layout the backstays were not used to support side spans, but were run by approximately right lines to the anchorages. This is the most economic layout possible, because a steel-trestle approach is always cheaper than any layout of truss spans that can be made, not only because it requires less metal, but also because the unit prices thereof erected are somewhat smaller.

The main piers of all the cantilever bridges and most of those for the suspension structures were designed as two pedestals with a reinforced-concrete wall between, this wall extending a short distance below extreme-low-water mark. It was found, however, in the case of the combined four-track-railway-and-highway suspension-bridges, that it was just as economic to use a continuous pier, because of the four points of support required by the tower columns, hence that feature of construction was adopted.

The method employed for finding the quantity of concrete in the anchor pier for a cantilever bridge was to compute the maximum uplift, multiply it by two, and divide the product by the weight of one cubic foot of concrete, taking due cognizance, of course, of the buoyant effort of the water on all submerged portions thereof. If the volume thus found would work up into a properly-shaped pier, well and good; but if not, an additional amount was provided.

The method of proportioning the anchorages for suspension bridges, when the foundations were solid rock, was to make each one quite long and narrow, high in the rear and low in the front, and to let the line of pressure reach the base exactly on the edge of the middle third thereof. In case the foundation were piles, a similar shape was used, but it was necessary to keep the load on each pile of the front row down to forty tons.

When piles were employed to support the main piers, the limiting load per pile was taken also at forty tons, exclusive of the effect of wind pressure. The piles used were all assumed to be one hundred feet long.

The limiting widths of structure were as follows: In cantilever bridges one twenty-fifth of the main opening; in suspension bridges one-twentieth thereof, measuring between central planes of exterior columns over main piers; and between central planes of stiffening trusses one-thirtieth of the main opening. As a matter of economy, in some of the cantilever structures the distance between truss planes was made as small as practicable for the suspended span, and was gradually widened out to a maximum over the main pier, and then gradually reduced to a minimum over the anchor pier.

The economic lengths for the cantilever structures were taken as established twenty years or more ago by the author when preparing the MS. of

"De Pontibus," viz.: For the suspended span, three-eighths of the main opening; for each cantilever arm, five-sixteenths of the main opening. As before stated, the length of the anchor arm, for the sake of appearance, was made the same as that of the cantilever arm, although some metal would have been saved by assuming it shorter.

In the suspension span, also, economic dimensions were used, viz.: one-fortieth of the length for the truss depth, and one-ninth thereof for the deflection of the cables. In order to provide proper splay for the latter (when splay was required), the tower width, as before indicated, was made one-twentieth of the main opening. This militated but slightly against the suspension bridge, because, in the substructure, it generally increased the cost of only the walls between the pedestals of the main piers, the increase being a bagatelle in comparison with the total cost of the said substructure.

The first estimates prepared by the author were for double-track-railway bridges; and he assumed, to begin with, an opening of 1,700 feet, which is approximately Dr. Steinman's span-length for equal cost. The profile adopted for this crossing is shown in Fig. 13*a* and Fig. 13*b*. It will be seen

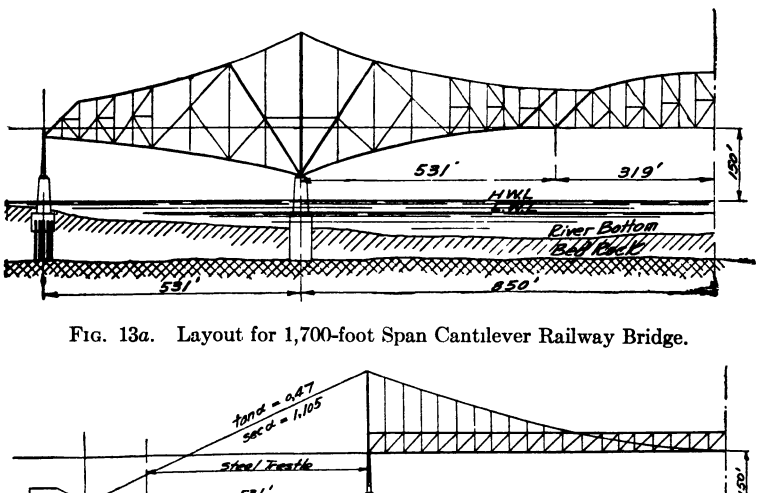

FIG. 13*a*. Layout for 1,700-foot Span Cantilever Railway Bridge.

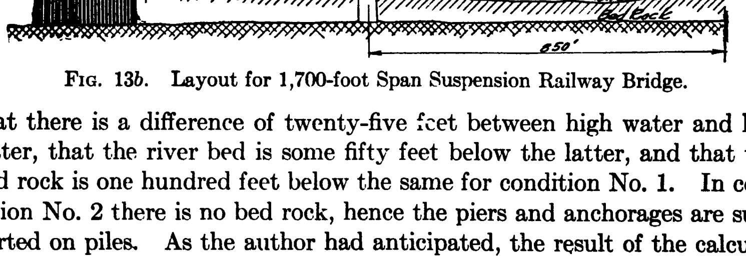

FIG. 13*b*. Layout for 1,700-foot Span Suspension Railway Bridge.

that there is a difference of twenty-five feet between high water and low water, that the river bed is some fifty feet below the latter, and that the bed rock is one hundred feet below the same for condition No. 1. In condition No. 2 there is no bed rock, hence the piers and anchorages are supported on piles. As the author had anticipated, the result of the calculations showed a large difference in favor of the cantilever structure, the

total costs being $4,120,000 and $7,980,000. These figures are for condition No. 1, in which all piers and anchorages were assumed to be sunk by the pneumatic process to bed rock.

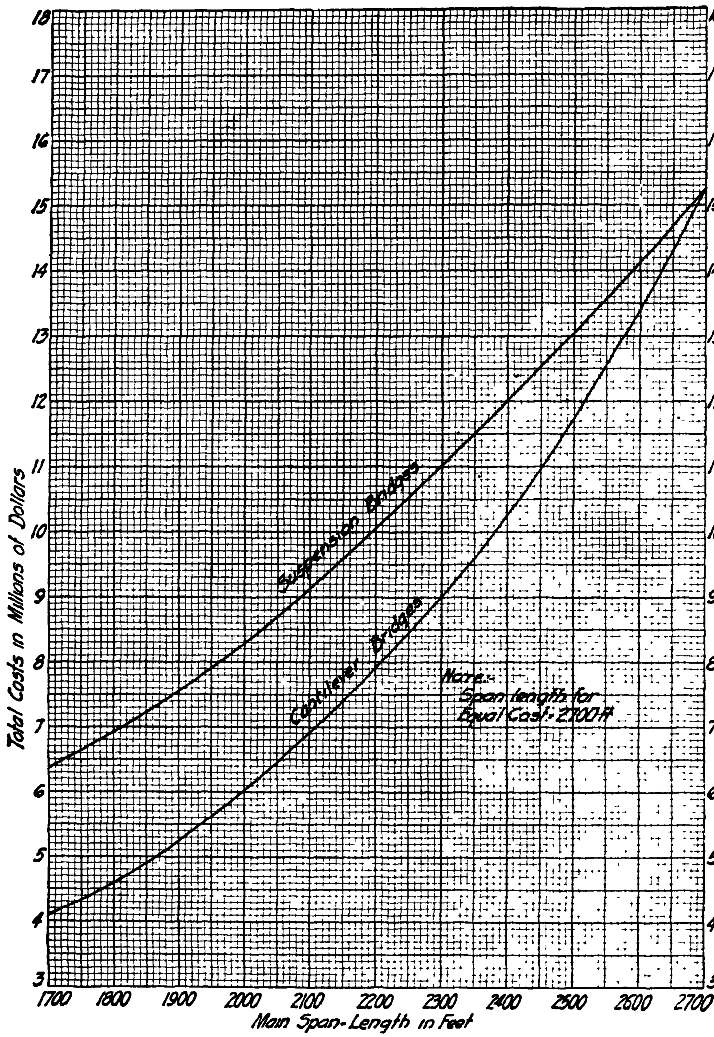

FIG. 13c. Cost Curves for Double Track Railway Bridges.

For condition No. 2, in which there is no bed rock within reach of the piles, the corresponding figures were $4,420,000 and $7,030,000.

The suspension-bridge anchorages resting on piles were found to be so much cheaper than those resting on bed rock that it was concluded to adopt them for condition No. 1, and to assume the piles to be driven to bed rock.

In order, however, to make the comparison perfectly fair, the anchor-piers of the cantilever bridge were also figured as resting on piles driven to bed rock. The result of this change was a large reduction of the difference in cost between the two types of structure compared, as shown by the following totals: $4,080,000 and $6,387,000. These are the costs which are plotted in Fig. 13c.

After noting the large difference in these total costs, the author decided to test a twenty-four-hundred-foot opening, thinking that surely for such a long span the suspension bridge would be the cheaper. The bed rock was kept at the same elevation as before. the only difference in the profile being that the width of river was increased, as shown in Fig. 13*d* and Fig. 13*e*. It was decided, in order to save labor, to do no further computing upon the basis of main piers resting on piles; but all anchor piers and anchorages were assumed to be thus supported, as in the final estimates for the 1,700-foot

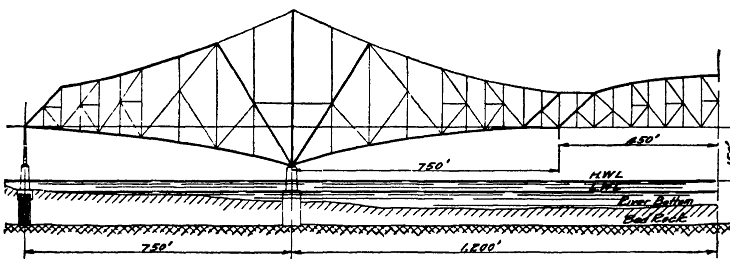

FIG. 13*d*. Layout for 2,400-foot Span Cantilever Railway Bridge.

spans. Much to the author's surprise, the results showed the cantilever structure to be still the cheaper, the total costs being $10,210,000 and $12,033,000.

Then an opening of twenty-seven hundred feet was tested, the result being $15,269,000 for the cantilever bridge and $15,259,000 for the suspension bridge. This shows that for double-track-railway bridges of nickel steel, the span-length for equal costs of cantilever and suspension bridges is 2,700 feet, or one hundred feet longer than the greatest advisable length for the former type recommended by the author in his paper on "The Possibilities in Bridge Construction by the Use of High-Alloy Steels."

In order properly to plot the curves in Fig. 13c, it was necessary to compute the cost of a cantilever bridge having a span of 2,050 feet. This gave four points on the curve and enabled it to be sketched in satisfactorily, after which it was easy to draw the corresponding curve for the suspension bridge.

In order to make as good a showing as practicable for the suspension bridge, as far as the layout is concerned, it was decided to assume that the bed rock comes quickly to the surface in the vicinity of the main piers and runs back thereafter at an elevation of about ten feet above high water in the manner adopted by Dr. Steinman. This assumption reduces greatly

the costs of the anchorages of the suspension bridges and to a much smaller extent those of the anchor piers of the cantilever structures. The effect of this change on the cost curves is shown in Fig. 13*f*. From it there will be observed that the span-length for equal cost has been brought down to about 2,570 feet, showing that the change made in the bed-rock profile has effected comparatively little variation in this span-length.

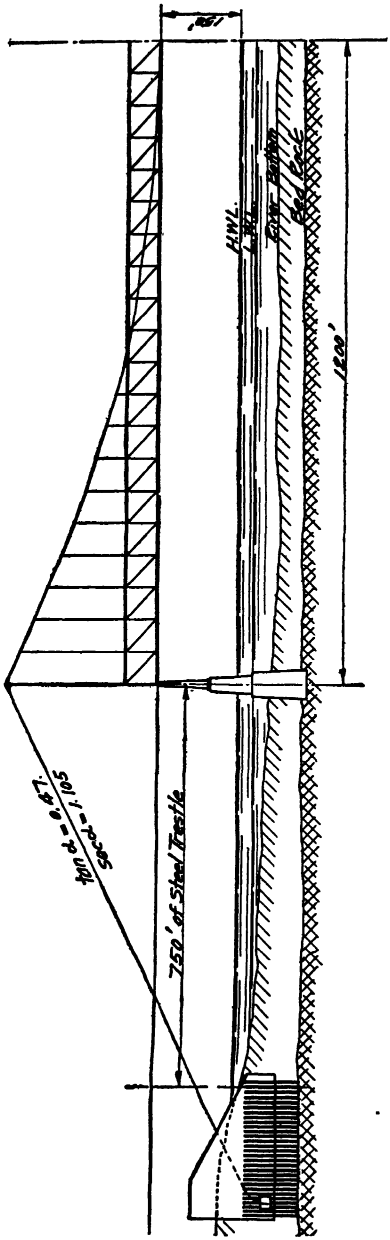

FIG. 13*e*. Layout for 2,400-foot Span Suspension Railway Bridge.

The result of the preceding calculations differs so fundamentally from that of Dr. Steinman that the author found it necessary to study carefully in detail the doctor's various assumptions and estimates, so as to discover the reason or reasons for the great difference—amounting to over one thousand feet. The following variations between his data and estimates as compared with those of the author were found:

First. In his cantilever bridges Dr. Steinman makes the ratio of length of suspended span to that of main opening vary from 0.5 for 1,000-foot openings to 0.4 for 2,000-foot openings, while the author two decades ago showed the economic ratio to be 0.375; and, as previously mentioned, he (Dr. Steinman) makes the length of the anchor arm 0.4 of the main opening instead of about one-half of that amount.

Second. Dr. Steinman's bridges carry both railway and highway live loads, while the author's are for railway traffic only.

Third. Dr. Steinman's suspension bridges have side spans supported by the backstays, while in the author's layouts these spans are replaced

by steel-trestle approaches entirely disconnected from the main structure.

Fourth. Dr. Steinman uses a tension intensity of working stress of 20,000 pounds for carbon steel and one of 30,000 pounds for nickel steel,

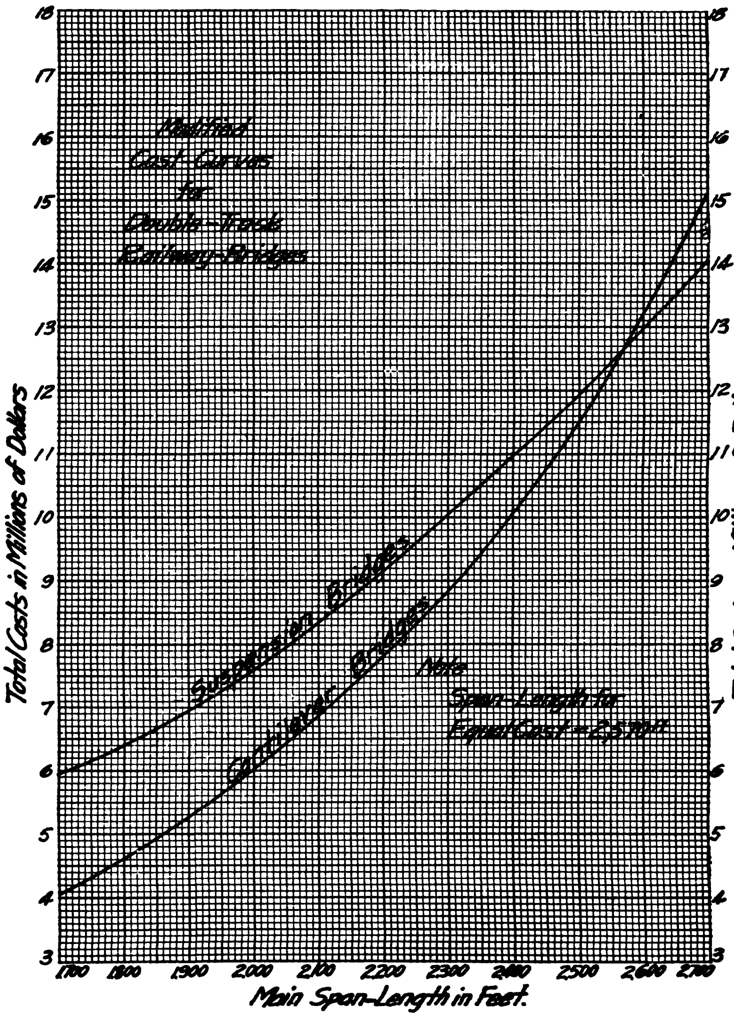

FIG. 13*f*. Modified Cost Curves for Double Track Railway Bridges.

while the author's practice has been to employ, respectively, 16,000 pounds and 28,000 pounds.

Fifth. Dr. Steinman ignores entirely the effect of impact on trusses, while the author allows for it. In the very long spans this cuts but little figure; however, such is not the case for the shorter spans.

Sixth. Dr. Steinman's estimated costs for substructure not only exceed greatly those of the author, but also the ratios of division thereof between main piers and anchorages are fundamentally different from his.

Seventh. In his cantilever-bridge estimates Dr. Steinman divides the metal into five groups, viz.: Suspended span, cantilever arms, anchor arms, towers, and anchorages, but some of the total amounts for these groups are greatly out of proportion.

Eighth. Dr. Steinman uses an intensity of working stress for wire cables varying with the span length, while the author has employed a constant value, in accordance with his standard practice of varying live loads and impact allowances and keeping the unit stresses unchanged. The effect of this variation would be to shorten somewhat the span-length for equal cost of the contrasted types.

A dissertation upon the first, sixth, and seventh variations may throw some light upon the subject; and, to make it properly, it became necessary to reproduce here Dr. Steinman's two layouts, as shown in Fig. 13*g* and Fig. 13*h*.

Is it not evident from a glance at Fig. 13*g* that the long anchor arms, passing over dry land, must be uneconomic as compared with steel trestlework, which, as is well known, is the cheapest kind of metallic structure? It is true that Dr. Steinman, Dr. Burr, and, possibly, other writers have shown mathematically that the economic length of the anchor arm is four-tenths of the main opening;* but such questions cannot be solved by mathematical analysis, for it is impracticable to consider by equations the many variables in the make-up of an anchor arm, as well as simultaneously a trestle approach. Dealing with this point, the author made the following statement in "De Pontibus": "When, however, the problem is to determine the economic length of anchor arm for a fixed distance between main piers, the result will be quite different; because, within reasonable limits, the shorter the anchor arm the smaller will be its total weight of metal, and because trestle approach is much less expensive than anchor arm. It would not, for evident reasons, be advisable to make the length of anchor arm less than twenty per cent of that of the main opening, or, say, fifteen per cent of the total distance between centers of anchorages. With this length there would probably be no reversion of stress in the chords of the anchor arm, even when impact is considered. Generally, though, the appearance of the structure will be improved by using longer anchor arms than the inferior limit."

If there were no other way to settle this question, the author would be willing to determine it beyond all possibility of doubt by preparing actual designs and estimates of quantities and costs for the anchor arm layout

* This is nearly correct for the case where the locations of the anchorages are fixed, while the main piers may be placed where desired; but in Dr. Steinman's study it is the main piers that are fixed in location, hence the assumption made for economic length of anchor-arms is unwarranted.

of Fig. 13*g*, and for a corresponding layout in which the exterior half thereof is replaced by steel trestle; but as this would involve considerable trouble

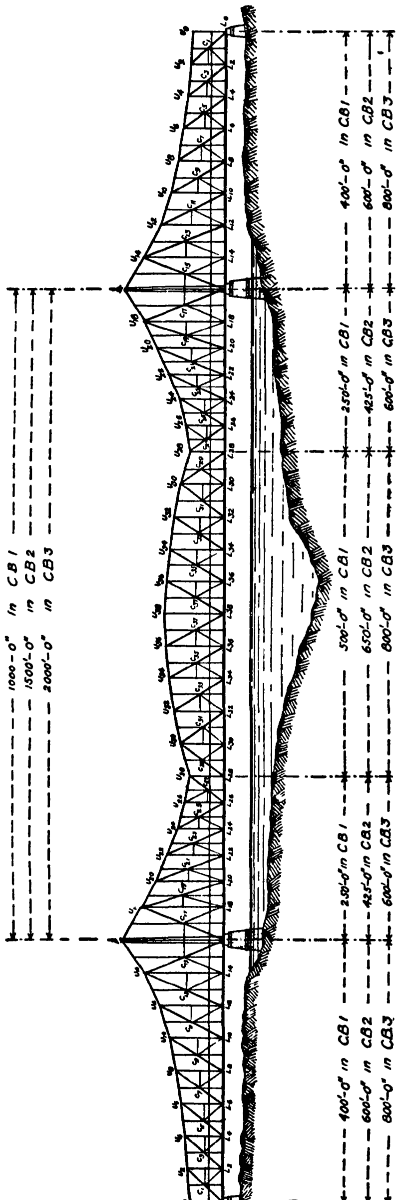

FIG. 13*g*. Dr. Steinman's Layouts for Cantilever Bridges.

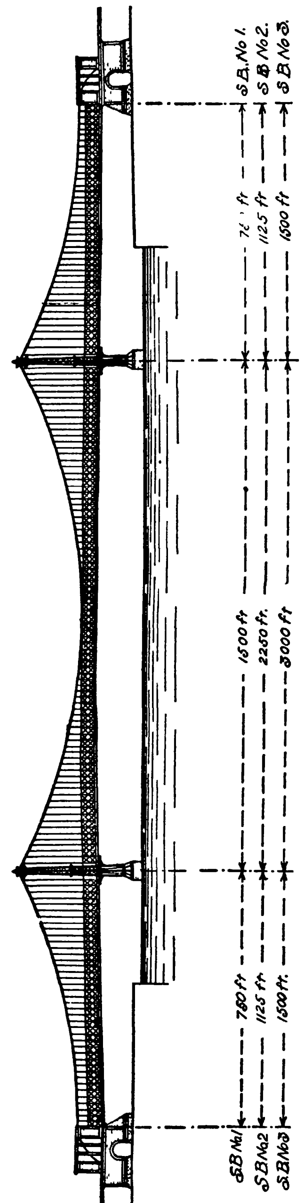

FIG. 13*h*. Dr. Steinman's Layouts for Suspension Bridges.

and an expense amounting to several hundreds of dollars, the following *a priori* observations ought to be sufficiently convincing:

First. About 29 per cent of the total weight of the trusses and laterals of this anchor arm is included in its outer half, and the average weight per foot of this portion is about 71 per cent of that for the entire structure. Applying this to the already computed weights of a double-track-railway cantilever-bridge having a 1,500-foot opening, and using the unit costs of materials in place as stated, makes the average value per linear foot of the outer half of the anchor arm $640. The weight of metal per lineal foot for a double-track steel-trestle one hundred and forty feet high is 4,200 pounds and its value is $210, to which should be added not to exceed $5 per lineal foot for the cheap concrete pedestals required to raise the column feet a short distance above the rock foundation. This shows that the trestle costs only one-third as much as does the outer half of the anchor arm.

Second. While it is conceded that the remaining portion of the anchor arm may weigh somewhat less per foot than it would as an independent arm, the difference will be small for the following reasons:

(*a*) As the moment over the pier is the same for all lengths of the anchor arm (because it comes entirely from the loadings on the cantilever arm and the suspended span), the weights of metal in the truss members lying near the pier will not differ greatly in the two cases.

(*b*) While the negative stresses due to the uplift will be increased by the halving of the resisting lever arm, on the other hand the direct live load stresses will be greatly diminished because of the halving of the span length, these two effects tending to offset each other.

(*c*) With the short anchor arm, the stresses in the outer diagonals (as well as in all the other main diagonals) and in the top chord members will always be tensile, hence eye-bars can be used for these members, thus effecting a great saving; because, owing to the increase in sectional area (to allow for rivet holes) and to the weight of the details, it takes nearly fifty per cent more metal to build a riveted tension member than is required for the corresponding eye-bars and their pins.

(*d*) While it is true that the short arm produces a greater uplift and, consequently, necessitates a heavier anchorage, it must be remembered that the value of an economically-designed anchor-pier is very small in comparison with the cost of the rest of the structure. Again, it must not be forgotten that with the long arm there is positive as well as negative loading on the anchor pier, and that, in consequence, it is possible that there would be no difference worth mentioning in the costs of the two anchor piers.

It seems to the author that, in view of the preceding, it ought to be evident without further calculation that a length for the anchor arm equal to two-tenths of the opening ought to be decidedly more economic than a length twice as great.

In respect to the substructure, Dr. Steinman in his design for his 1,500-foot-span, four-track, steam-railway-and-highway, cantilever bridge found the cost of two main piers to be $1,262,000, and that of two anchor piers

$1,032,000; while the author found for his nearest corresponding double-track, steam-railway bridge $827,000 and $161,000. While it is entirely impracticable to compare these figures, because of fundamental differences in both the loading and the foundation conditions, it is evident that Dr. Steinman must have made some serious mistake in his calculations when he caused the costs of his main piers, with their pneumatic foundations, and his anchor piers, resting on bare, dry bed-rock, to be so nearly alike; because the latter generally are insignificant affairs when compared with the former. This same error exists in the other two cantilever bridges which he has computed; for in his 1,000-foot span he found, respectively, $876,000 and $524,000, and in his 2,000-foot span $2,153,000 and $1,994,000. This matter will receive additional attention later on.

In respect to his division of weights of metal in superstructure, Dr. Steinman recorded the following:

TABLE 13ι.

WEIGHTS OF METAL IN POUNDS

Main Span in Feet	The Suspended Span	Two Cantilever Arms	Two Anchor Arms	Two Towers	Two Anchorages
1,000	8,738,000	6,551,000	9,697,000	5,987,000	785,000
1,500	15,550,000	16,951,000	20,566,000	17,479,000	1,794,000
2,000	28,964,000	39,750,000	42,851,000	40,158,000	3,374,000

Referring to the item of weight of the towers in both the 1,500-foot-span and the 2,000-foot-span structures *it exceeds the total weight of metal in the cantilever arms, and is but little less than that in the excessively-long anchor arms.* Surely this cannot be correct! Each tower consists of two braced columns, the load on each of which is composed of the vertical components of the stresses in the two upper-chord members meeting at its top, and these are not extraordinarily great. Had the upper chords been run horizontally from inner hip to inner hip, the column stresses would have been zero, barring those due to their own weight and to an insignificant wind pressure on the columns themselves only.

Such glaringly-great irregularities as these upset the entire economic comparison and render its results worthless. Moreover, all these variations from correctness combine to militate against the cantilever structure. On the other hand, though, the assumption of side spans supported by the backstays militates against the suspension structure.

In view of the preceding, the author concluded that it would be necessary to compute quantities and plot cost curves for cantilever and suspension bridges of the type and loading assumed by Dr. Steinman, adhering as closely as practicable to his general features of layout, character of metal used in the various parts, weights per foot of floor systems and lateral sys-

tems, and cost per foot of trestle approaches; but differing with him in the following particulars:

First. Raising the grade of the structure so as to afford a vertical clearance of 150 feet above high water, and lowering the elevation of main pier foundations to 35 feet below low water. This is more in accordance with probable actual conditions than is indicated by the profile in Fig. 13*g*.

Second. Substituting steel-trestle approaches for the side spans shown in Fig. 13*h*.

Third. Adopting the most economical type of substructure for each case.

Fourth. Adopting a length of anchor arm equal to 5/16 L instead of 0.4 L.

With these premises the author computed the costs of cantilever and suspension bridges for openings of 1,500 feet and 2,400 feet, and found that the span of equal cost lies between these limits. Then he figured for a 2,000-foot span. This gave him three points on each curve, as shown in Fig. 13*i*, besides which, he estimated in detail by proportion the costs for several other openings and plotted the results of these also. Fig. 13*i* shows that the span-length for equal cost is about 2,190 feet, instead of the 1,670 feet found by Dr. Steinman—a difference of over 500 feet.

It will be interesting to compare the substructure costs found by Dr. Steinman and those found by the author for like spans and practically the same general conditions, as recorded in the following table:

TABLE 13*b*

Main Span-length and Character of Structure	Cost in Dollars of the Main Piers		Cost in Dollars of the Anchorages	
	Steinman	Waddell	Steinman	Waddell
1,000-foot Cantilever........	876,000		524,000	
1,500-foot Cantilever........	1,262,000	506,000	1,032,000	290,000
1,500-foot Suspension.......	1,330,000	440,000	2,428,000	1,222,000
2,000-foot Cantilever........	2,153,000	660,000	1,944,000	310,000
2,000-foot Suspension.......	.. .	600,000		1,920,000
2,250-foot Suspension.......	1,835,000	....	4,174,000	
2,400-foot Cantilever.......	....	926,000	. .. ..	357,000
2,400-foot Suspension.......	....	880,000	. ..	2,400,000
3,000-foot Suspension	3,017,000	....	6,995,000	

Of the preceding nine cases there are only three which can be directly compared, viz., the 1,500-foot cantilever, the 1,500-foot suspension, and the 2,000-foot cantilever, although other comparisons might be made approximately by interpolation. It will be noticed that Dr. Steinman's main piers cost two or three times as much as those of the author, his anchor piers for

cantilevers from three and a half to six times as much, and his anchorages for suspension bridges about twice as much. This gives further proof of the statement previously made to the effect that his entire economic investiga-

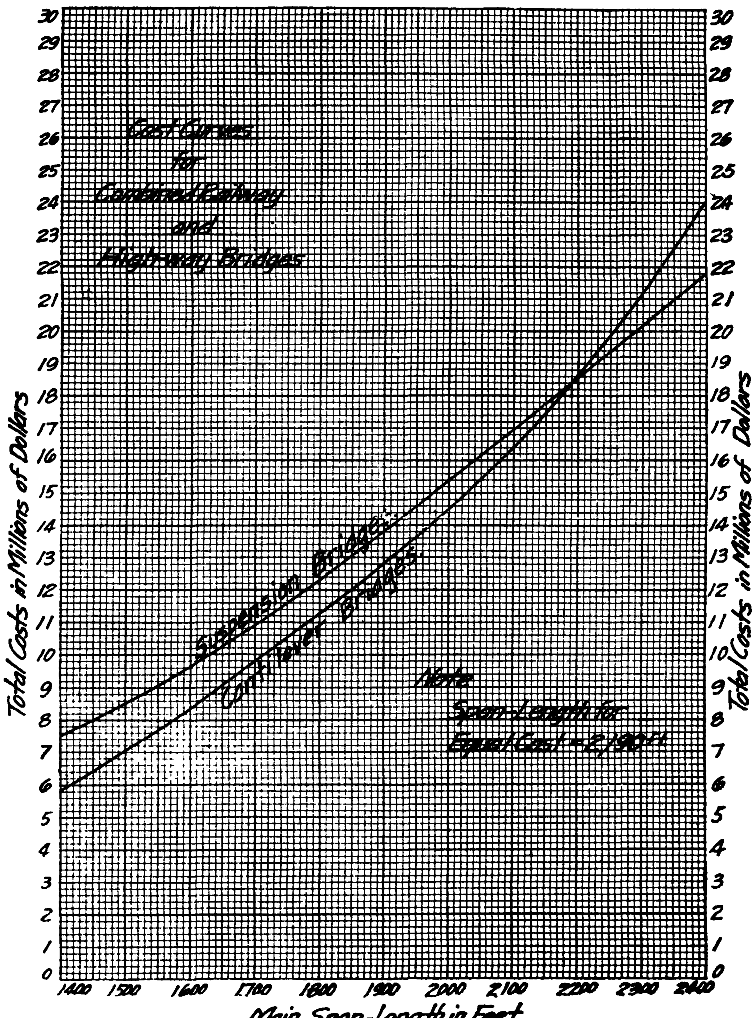

FIG. 13*i*. Cost Curves for Combined Railway and Highway Bridges of the General Type Computed by Dr. Steinman.

tion is incorrect and that the deduction which he makes therefrom concerning the span length for equal cost is wrong.

In thus criticizing Dr. Steinman's little book, the author does so merely because he feels that the profession should not be left in error on such an

important point as the comparative economics of cantilever and suspension bridges. Some time in the not very distant future there are going to be built in this country many long-span bridges; and it behooves engineers to know in advance the economics of the different types of structures applicable thereto. Dr. Steinman deserves great credit for his energy and courage in attacking such a stupendous problem at such an early date in his professional career, without any records of weights at his disposal, and before he had had any actual experience in bridgework. In undertaking such an immense task he set a splendid example to other young engineers; and the incorrectness of his conclusion is no blot whatsoever upon his professional record. It would be well for engineering if there were in its ranks many more young men possessing the attributes of energy, ambition, and love for hard work to the same extent that he does. Such men will be badly needed in every branch of technics, if our profession is to take the high position in the community to which it is entitled by its importance to mankind.

Dr. Steinman can console himself with the reflection that he is not the only engineer who has devoted an entire treatise to the production of a wrong conclusion, for several decades ago an eminent French professor of engineering published a large book dealing with the economics of truss bridges, basing his calculations upon such incorrect premises that the result of his work was of no real value to the profession.

Comparing the results of the preceding calculations, as shown in Fig. 13*c* and Fig. 13*i*, it will be noticed that the span length for equal cost is much less for the combined railway and highway type of structure than for the strictly railway type. The reason for this is that in a modern highway bridge the proportion of dead load to live load is much greater than it is in a railway bridge, because of the large weight of the pavement, the supporting slabs, and the concrete footwalks. In the stiffening trusses of a suspension bridge it is generally the live load only which causes stresses that influence the sectional areas of the members, the dead load having no effect thereon whatsoever, but in a cantilever bridge it is the total live load plus the dead load which does so, with sometimes a little assistance from the wind load; hence it is evident that the smaller the proportion of live load to total load the more favorable it is for the suspension bridge. On this account, in strictly highway structures, the span length for equal cost will be much shorter than those thus far determined. Not knowing what the length would probably be, the author figured the costs of the two types for 1,500-foot, 1,200-foot, and 1,000-foot main openings, using carbon steel only; and from the results of the computations he plotted the curves in Fig. 13*j*. From this diagram it will be seen that the span length for equal cost is about 1,000 feet.

Recognizing that this investigation would not be complete without preparing a set of computations for strictly-highway structures of nickel steel, the necessary calculations were made for a 1,000-foot span of each

type, the result showing almost exactly equal costs. This indicates that the strength of the steel used does not modify the span length for equal cost in highway structures, although changing the totals of the estimates.

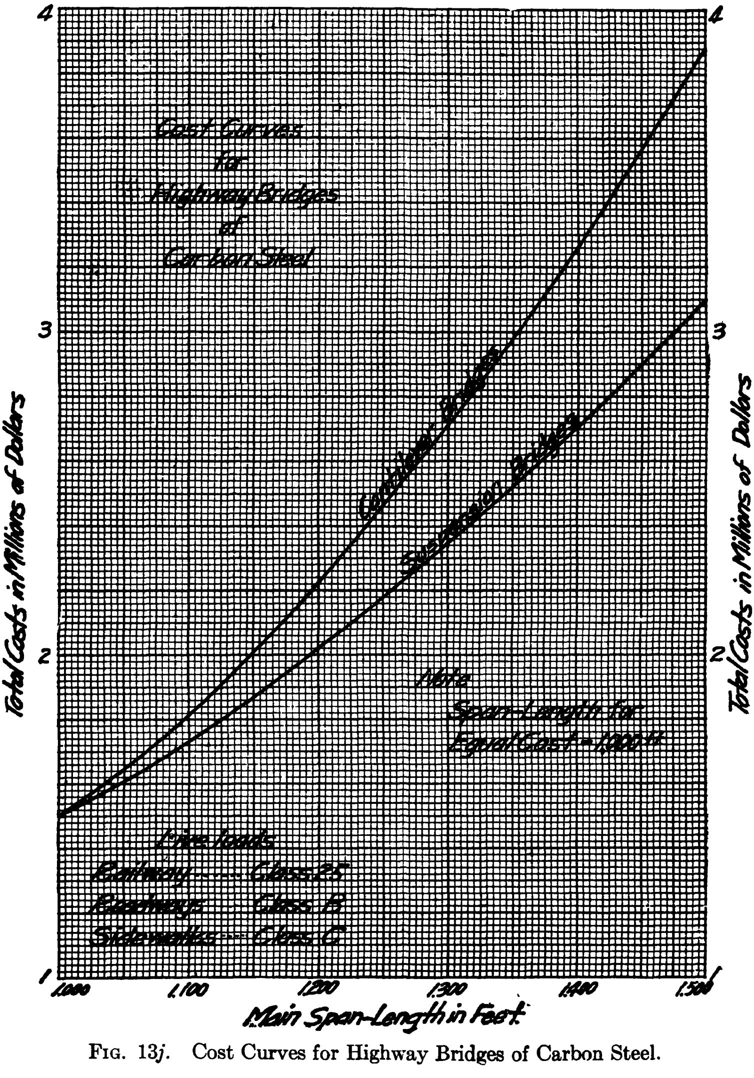

FIG. 13j. Cost Curves for Highway Bridges of Carbon Steel.

These highway bridges are of the same kind as that adopted as standard by the author in his late paper on "The Economics of Steel Arch Bridges," viz., a deck about 60 feet wide, out to out, composed of a paved roadway 42 feet wide, resting on a reinforced-concrete base, and having a double-track street-railway at the middle, and two 8-foot-wide, reinforced-concrete

sidewalks. The live loads for the floor system are Class 25 for the electric railway, Class B for the rest of the roadway, and Class C for the sidewalks. Class A over the full width of the deck was employed for the trusses.

RÉSUMÉ OF INVESTIGATION

First. For exclusively railroad bridges, the economic limit for the cantilever type of structure, or, in other words, the main span length for a cost equal to that of the corresponding suspension bridge is that length which requires $4\frac{1}{2}$ pounds of metal to carry 1 pound of live load.

Second. For modern highway structures, carrying also incidentally electric railway tracks, this span length for equal cost is 1,000 feet.

Third. For combined-railway-and-highway structures the limit is intermediate between the limit for railway structures and that for highway structures, the interpolation being done in direct proportion to the ratio of railway-truss-live-load to total-truss-live-load.

This may be expressed by formula thus: If G is the span length of equal cost for strictly-railway bridges, and R is the ratio of railway-truss-live-load to total-truss-live-load, then, for combined-railway-and-highway structures the span length for equal cost will be given approximately by the equation:

$$S_c = 1{,}000 + (G - 1{,}000)R$$

For instance, if $G = 2{,}700$ feet for nickel steel railway bridges and $R = \frac{2}{3}$,

$$S_c = 1{,}000 + 1{,}700 \times \tfrac{2}{3} = 2{,}133$$

This checks fairly well with the value shown in Fig. 13*i*, where

$$R = \frac{12{,}000}{18{,}000} = \frac{2}{3}.$$

Fig. 13*k* is a diagram from which can be found at a glance the span length for equal cost for any proportionate combination of railway and highway live loads, under the assumption that nickel steel is employed for the principal portions of the structure. In case, though, that carbon steel alone be used, which is unlikely, the limiting span length for cantilever construction is to be taken at about 2,000 feet.

While it was not intended to do any figuring concerning the comparative economics of cantilever and suspension bridges when alloy steels having higher elastic limits than 60,000 pounds per square inch are employed, it was surmised that the span length for equal cost for strictly-railway bridges will not differ essentially from the limiting lengths for cantilever main spans determined by the author in "The Possibilities in Bridge Construction by the Use of High Alloy Steels," viz.:

For $E = 70{,}000$ pounds per square inch	2,780 feet
For $E = 80{,}000$ pounds per square inch..	2,910 feet
For $E = 90{,}000$ pounds per square inch..........	3,030 feet
For $E = 100{,}000$ pounds per square inch.......	3,140 feet

It is possible that the author is not entirely justified in making this surmise, because computations might show that the span length for equal cost does not exceed 2,700 feet, no matter how high may be the alloy of steel used. It seemed hardly worth while to spend much time in figuring

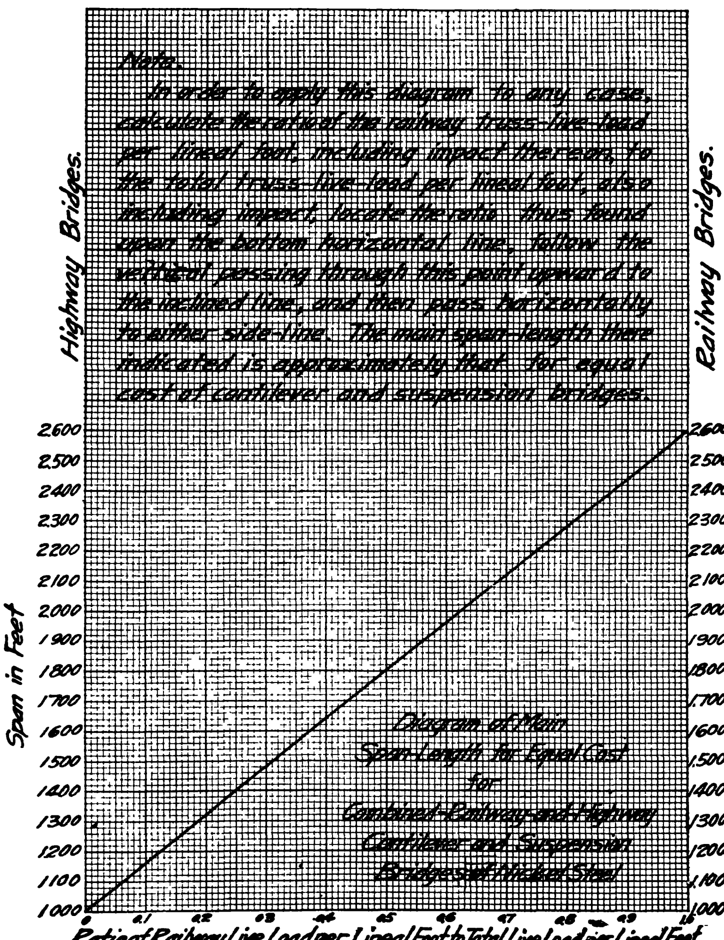

FIG. 13k. Diagram of Main Span Lengths of Equal Cost for Combined Railway and Highway Cantilever and Suspension Bridges.

upon this question before a high alloy of steel satisfactory for long-span-bridge building is found; nevertheless, as a matter of curiosity, it was decided to test a main span length of 2,900 feet, for steel having an elastic limit of 80,000 pounds per square inch, and assuming that the cost of that metal in place is 9 cents per pound.

The comparative figures of cost for the two structures proved to be as follows:

Cantilever bridge.	$15,720,000
Suspension bridge.	15,233,000

However, had the price of the alloy steel been taken at 8 cents per pound the same as for nickel steel, the cost estimates would have been as follows:

Cantilever bridge.	$14,448,000
Suspension bridge.	14,856,000

As these last figures reverse the previously found economics of the two types, it is evident that for bridges of high-alloy steels the span length for equal cost is vitally dependent upon the pound price of the said alloy steel, the lower it is the more favorable is it to the cantilever structure. In view of the fact that at present no one has any idea of what the cost per pound will be for high-alloy steels used in future long-span-bridge construction, it will be well to adopt temporarily as correct the author's before-mentioned surmise, viz., that in alloy steel bridges carrying railway loads only, the span length for equal cost is that for which, in the cantilever bridge, there are required $4\frac{1}{2}$ pounds of metal to sustain 1 pound of live load.

The author recognizes that a change in the assumed conditions would modify somewhat all the previously found span-lengths of equal cost for both carbon-steel and nickel-steel bridges; but he does not believe that the variation will be material—say not to exceed 2 or 3 per cent in any case for any one fundamental change, or 5 per cent for any probable combination of changes. For instance, if the main piers rest on piles instead of going to bed rock, this will militate a little against the suspension structure, increasing slightly the span length for equal cost. The same effect occurs if the pound price for steel cables be increased without changing the pound prices for the other metals, and vice versa.

If the unit prices for substructure be decreased, the result will be favorable to the suspension bridge, because, while the main piers will be affected about alike, there will be a greater saving in the anchorages of the suspension bridge than in the anchor piers of the cantilever structure. Let us see what effect it would have to reduce the prices of all concrete work five dollars per cubic yard, thus bringing them close to the lowest limits for truly-first-class construction that have existed in periods of national depression.

In the railroad bridges of 2,700 feet span, the reduction in total cost of substructure would be $473,000 for the cantilever bridge and $928,000 for the suspension bridge, making the total costs, respectively, $14,796,000 and $14,330,000. Performing the corresponding reduction in prices of substructure for the 2,400-foot spans gives, for the total costs, respectively, $9,877,000 and $11,196,000. Plotting these points on a cross-section

diagram and joining them properly by very slightly curved lines shows that the span length of equal cost is reduced from 2,700 feet to 2,640 feet. This is no material amount, indicating, as it does, a variation of only 2.2 per cent.

Addendum

The preceding was written in the summer of 1918. A year later the author was called in by some prominent citizens of Detroit to make a study of the governing conditions for a proposed highway-and-street-railway bridge over the Detroit River, practically on a line joining the business centers of the cities of Detroit and Windsor, and to determine upon the best type of structure to adopt. A few days of investigation led to the conclusion that a single span of 2,500 feet, crossing the entire river in the clear between harbor lines, would be obligatory; and, accordingly, the layout and the approximate cost-calculations were made for a suspension bridge. It became necessary to obtain pound prices for structural metal (both nickel steel and carbon steel) and wire cables in place; and the following values were found:

Carbon steel erected	7.0¢ per lb.
Nickel steel erected	9.0¢ per lb.
Cables erected	23.0¢ per lb.

The last figure was simply staggering! Surely, such an enormous price can be only temporary, for the great difference between it and the other two figures is altogether illogical. Nevertheless, it shows the possibility of an abnormal price-condition existing long enough to affect temporarily the economics of cantilever and suspension bridges. It is not likely that there can ever be a worse condition than the one at present governing; consequently, the author has recast for existing unit prices the estimates of cost made for the preceding investigation, and has found the following results:

The span of equal cost for highway bridges has been advanced from 1,000 feet to exactly 1,200 feet; that for the particular combined bridges investigated has been increased by 170 feet; but that for the steam railway bridges has been augmented only 60 feet. The reason for the smaller increase in the last case is that, in cantilever structures the weight curves, and consequently the cost curves, rise very rapidly at a span of 2,700 feet, because such a length is really a little beyond the truly practicable limit for that style of bridge.

These variations are somewhat greater than the maximum which the author anticipated when writing his paper; but at that time he never would have deemed it possible that such a great variation in unit prices of structural steel and wire cables could hold as that which exists to-day; nor does he now consider it possible that it can be made to last for any great length of time.

In making the Detroit-Windsor Bridge study, a practical proof was given of the usefulness of the paper. No copy of "Bridge Engineering" was available for making an estimate of the cost of the suspension bridge and its approaches, but a copy of the paper was at hand; and, as a rough estimate was required immediately, the following procedure was adopted, it being recognized at the outset that all the assumptions made therein were upon the side of safety, and that, consequently, the resulting figures of cost would be somewhat too great:

Referring to Fig. 13*j*, the curve for costs of suspension bridges was extended on an enlarged cross-section sheet to a span of 1,700 feet, at which length the spans on Fig. 13*c* begin. The cost thus found was multiplied by the ratio of the total combined clear widths of roadway and sidewalks for the two structures considered, and the product was multiplied by the average of the ratios of the unit costs of all substructure and superstructure materials in place for present conditions and the conditions assumed in the paper. Then, referring to Fig. 13*c*, it was noted that the cost of a 2,500 foot-span suspension-bridge and its approaches is almost exactly double that for a similar 1,700-foot span with its approaches; hence the cost just found was doubled, and to the result were added the cost of the entire flooring from entrance to exit of structure, an allowance for the greater length of the approaches involved, and the approximate cost of either elevators or an escalator and a stairway at the Detroit approach.

Later, a more exact estimate of cost was made from the various data in "Bridge Engineering," the result being some 5 per cent less than that of the first approximation. This more-exact estimate was computed in a single working day. Without the aid of the book mentioned, it would probably have required as many weeks of figuring as it actually took hours thereof, in order to obtain results of equal accuracy.

In the Appendix to the original paper there are given five pages of estimates of cost, covering fourteen structures out of the twenty-five that were computed. It has not been deemed worth while to reproduce them in this treatise; for probably they would not be of much interest to any reader. If, though, anyone desires to see them, he can do so by consulting the *Transactions* of the Western Society of Engineers.

CHAPTER XIV

ECONOMICS OF BRIDGE APPROACHES

THE economics of approaches to bridges will involve the question whether it is best and cheapest to build earth embankments, timber trestles, steel viaducts, reinforced-concrete viaducts, or any combination of these, and at what heights it would pay to change from one type of construction to another.

In determining the economics of the different kinds of structure it does not suffice to compare merely their first costs; for it is necessary to take into account the items of depreciation, maintenance, and repairs by computing the annual expenses for these, finding the sums of money which, at the governing rate for simple interest, would produce these annual amounts, and adding the results to the first costs.

In certain cases it might not be best to adopt the theoretically-economic kind of structure, because the requisite funds for building it may not be available; and in such cases the cost of renewals should receive due consideration by taking cognizance of the probable increase in the future prices of perishable materials, such as timber, as well as of the special danger to the structure from fire or washout due to the employment, either permanently or temporarily, of such inferior construction. As indicated in a previous chapter, the danger from fire to a structure built either wholly or partially of timber is a serious matter. It may be permissible under certain conditions to risk losing an approach to a bridge by either fire or flood; but if the danger extends also to the main structure, the cheapening expedient is not permissible.

Again, due consideration should be given to the question of the expense caused by the interruption of traffic by putting out of commission either one or both of the approaches. Generally speaking, it does not pay to take any chance of even temporary disaster to the structure; but, as before pointed out, it sometimes appears to be unavoidable.

In the case of embankments when earth is expensive at the outset and can be brought to the site much more cheaply after the bridge is finished and the railroad line that it carries is in operation, it will generally pay to build, as inexpensively as possible, a timber trestle; and later, just before it begins to need expensive repairs, fill around it and construct an embankment by dumping earth from above by means of a construction train.

There is an economic question concerning embankments, not at all difficult to settle, which exists when the right-of-way is expensive; and that is whether it is preferable to use wide banks with the natural side-slopes or to build concrete side-walls and thus diminish the area to be occupied. The only proper way to determine the economics in this case is to make a complete estimate of cost for each layout, based upon current prices of materials, labor, and right-of-way.

Occasionally in an engineer's practice there arises the economic question whether it will be better to build an expensive abutment with wing-walls, and possibly also toe-walls, or an inexpensive buried pier with larger superstructure and with rip-rap protection along the end and sides of the embankment. In most cases the latter will prove the more economic, but that such is the case should never be assumed without making accurate comparative estimates. With substantial bank protection that no flood is likely to wash out, the expedient of the buried pier is a perfectly legitimate one, and the construction involved by its use can properly be deemed first-class.

The choice between a steel trestle and a reinforced-concrete trestle for an approach should always be determined by including in the comparing estimates of cost the equivalents for depreciation, maintenance and repairs, giving a substantial preference to the concrete layout because of the possibility of future deterioration of the steel due to neglect of painting.

As indicated on page 1193 of "Bridge Engineering" there are given in Chapters 53, 55, and 56 of that treatise a large number of tables and diagrams, by means of which can be quickly computed the costs of the embankments, timber trestles, steel viaducts, reinforced-concrete viaducts, retaining walls, abutments, and culverts which may be needed in estimating the cost of approaches to bridges. From these data there can also be found very easily the comparative economics of plain and reinforced concrete for building retaining walls and abutments.

It is sometimes the case that in the approaches to a proposed bridge there would be a variation in the total cost of right-of-way and property damages by adopting different kinds of construction therefor, hence this matter should always receive due consideration. One of the most effective methods of economizing on these items is to substitute a spiral approach, such as mentioned on page 1076 of "Bridge Engineering," for the usual straight trestle. While the construction costs of the two types do not differ materially, if the property occupied or damaged be very valuable, a great saving can sometimes be secured because of the comparatively small and compact area required for the spiral. Moreover, it is sometimes practicable to construct a building in connection with the latter, that will bring in such large rentals as more than to wipe out all costs for right-of-way and property damages.

In general, it may be stated that timber trestle is the cheapest kind of approach, as far as first cost is concerned, excepting for small heights, but

that its upkeep and replacement are expensive. If funds for the construction are limited, it may be best to adopt timber trestle-work in spite of its being ultimately uneconomic, with the expectation of saving from the traffic receipts enough money to substitute later on, when replacement becomes necessary, the most desirable type of construction.

Of the permanent types of approach, the embankment is the cheapest where the property-cost is little or nothing, excepting when the grade line is very high or the earth difficult to obtain and, therefore, expensive.

When property is costly or side slopes are not permitted, it is economic for comparatively-low grade-levels to adopt earth embankment between retaining walls.

As the height increases, it becomes cheaper to pass from embankment to trestlework; and the point of division is not difficult to determine when there are no side walls, but when these are requisite, it will be necessary, as previously mentioned, to make the determination by actual cost-estimates. The wider the approach is at grade surface, the greater, for economy, will be the limiting height of embankment.

For low trestles of permanent construction, reinforced-concrete will generally prove economic, but for high ones it will be found necessary to employ steel.

CHAPTER XV

DETERMINATION OF LAYOUTS

THE determination of the best possible layout for any proposed structure is truly an economic problem, notwithstanding the fact that many of the considerations which influence it may not bear directly on the question of cost. It is one of the most important responsibilities in the province of the bridge engineer, and to do the work in the most effective manner possible demands a wide experience, coupled with good judgment and the ability to foresee eventualities over a long period of years. The general idea that the best possible layout is the one which makes the first cost of structure a minimum is a fallacy; for there are many other considerations besides economy in initial expenditure that are of great importance. The following is a fairly complete list of the various items which should be carefully considered before settling finally upon the layout of grades, clearances, span-lengths, character of substructure, and type of superstructure to adopt. This is a long list, but it must be remembered that it is intended to cover all the considerations for all cases, and that, probably, only a few of the items will apply to any particular case.

LIST OF FACTORS AND CONDITIONS AFFECTING THE LAYOUTS OF BRIDGES

A. Government Requirements.	I. Stream Conditions.
B. Grade and Alignment.	J. Foundation Considerations.
C. Geographical Conditions.	K. Navigation Influences.
D. Commercial Influences.	L. Construction Facilities.
E. Property Considerations.	M. Erection Considerations.
F. General Features of Structure.	N. Æsthetics.
G. Future Enlargement.	O. Maintenance and Repairs.
H. Time Considerations.	P. Economics.

While there is an attempt at logic in the arrangement of the preceding list on the combined lines of natural sequence and comparative importance, it is impossible to state in advance for any particular case or class of cases which are the items that should receive the most consideration. Each item will be taken up and discussed in the order adopted in the list.

GOVERNMENT REQUIREMENTS

In Chapter L of "Bridge Engineering" the requirements of the United States Government regulating the bridging of navigable streams are treated

at length. Neither the Federal Government nor any of the State Governments, however, concern themselves with the bridging of streams that are not navigable, unless it happen that suit against the builder or the proposed builder of the bridge be instituted in either a State or a Federal Court, when, of course, the law will be concerned.

The War Department nearly always confines its attention to a few salient features of any proposed crossing of a navigable stream, viz., the span-lengths, the clear waterway for navigation, the angle of skew (if the crossing be not square), the position of the movable span or spans (if there be any), the clear headway above high water for both the movable and the fixed spans, the character and the dimensions of the draw protection, and the amount of obstruction to the flow of water caused by the piers—especially those parts thereof below low-water mark.

In spite of the fact that the War Department has certain rules for determining the span-lengths for crossing various navigable rivers, the said rules are more or less elastic; hence it will generally pay any consulting bridge engineer, or other engineer who intends to bridge navigable water, to consult first with the local engineer of the Government who has charge of the district in which the proposed structure is located, and later, if necessary, with headquarters at Washington, in order to settle as to what the exact requirements of the Government will be. Often by stating one's case clearly and logically one can persuade the authorities to ease up on some regulation that appears to be unnecessarily strenuous or severe. For instance, the relation between the widths of clear openings required for swing spans and bascules or vertical-lift spans is a matter that has never been finally determined by the Department, each case as it arises being solved on its own merits.

Again, if the limiting length of span set by the Government does not exactly fit a crossing, one has to put in a shorter span at one end of the bridge, or to increase equally all the span-lengths, or else to obtain permission to decrease them equally. If the decrease be small, it is sometimes practicable to obtain the consent of the Department to the adoption of the shortened span-length.

In the case that the grade of a bridge is so low as to bring the clearance line too close to the elevation of high water to meet the Government requirements, it is sometimes possible to persuade the Department to permit an encroachment; but to do so would certainly be bad policy, for the limit set by the United States Engineers is adjusted about right to provide safety from passing drift.

In respect to the position of the movable span, the broad statement can be made that its mid-length should coincide with the deepest part of the channel, but there are occasional exceptions to the rule, notably when the channel is not permanent, or where it can advantageously be shifted by a little dyking. Permission to do such shifting and to locate the movable span accordingly would have to be obtained from the War Department.

The latter may have something to say about the angle of skew, as the United States Engineer Corps always advocates a square crossing, if it be practicable; hence the bridge engineer who desires to obtain approval for a bridge on a skew of any magnitude must be prepared to show good reason for his request; and even then it may not be granted, because, like the author, the Government engineers look upon a skew bridge as an abomination.

While the Department does not pay much attention to the character of the draw protection, it is likely to insist that it be not omitted and that its dimensions be satisfactory.

Ordinarily, also, it does not concern itself with the dimensions of the substructure; but sometimes, especially in case of a skew bridge, objection is raised to placing too much rip-rap around the piers and thus obstructing the flow of water in the channel.

Grade and Alignment

In most cases the grade and the alignment of the railroad or travelway are determined before the bridge engineer is called in, but sometimes it is otherwise; and there arise occasionally conditions which compel a conscientious bridge specialist to insist upon a change in either the grade or the alignment—or in both.

The ideal way to adjust the grade on a structure is to carry it over unbroken and, preferably, level in the case of railroad bridges, thus avoiding either a sag or a hump, as either of these objectionable conditions involves loss of power due to the climbing of unnecessary grades. Again, any great sag causes traction stresses and a shock that might better be avoided, if practicable. In a highway bridge this is not so important, and a rise from ends to centre of structure is permissible, especially as it facilitates drainage and improves appearance, notably in long-span suspension-bridges.

The ideal alignment for a structure is not only to have it on tangent throughout its entire length, but also to continue the said tangent quite a distance away from the bridge at each end. Sharp curves constitute an invitation for derailment; and a derailment on a bridge, or near the end of one, is liable to prove disastrous. A reverse curve on a structure, or on an approach thereto, is not permissible, if it can possibly be avoided. Where two curves in opposite directions come close together, there should be a stretch of tangent between them; and when this tangent is on a bridge, it should be made as long as possible. Sometimes it is entirely impracticable to avoid curvature on bridges and their approaches, as in the case of a railroad following the course of a river that runs between high banks and having to cross it from time to time in order to avoid heavy excavations and tunneling. In such cases curves on the approaches are unavoidable, and often it is necessary to put a part or even the whole of the structure itself on curve.

Such a general condition existed on the line of the Canadian Northern Pacific Railway as it followed up the Fraser and the Thompson rivers, crossing them nine times with only one structure entirely on the square.

In some skew crossings, especially when the obliquity is small, it is permissible to square the piers to the structure, thus saving considerable masonry; but this practice is not always advisable because of the damming of the water by the large area of the substructure that is opposed to the current

The layout of any bridge on a curve, or which has its approaches on curve, is greatly affected by the curvature, in that it has a tendency to shorten the span-lengths in the effort to avoid excessive width of superstructure and undue increase in length of piers.

The determination of the best grade to use for the approaches to a bridge is an economic problem of major import. It is of much more consequence in railroad bridges than in highway structures, because of the far steeper grades which are permissible in the latter; for, of course, the steeper the grade the shorter the approach and the less its cost. It is sometimes practicable to put a grade on the river spans of a bridge, leading up to a movable span or to a single, high-level channel-span; and this should always be done when practicable, notwithstanding the fact that the grade may have to be less than that allowable for the approaches, because even inches in elevation on the land construction often count materially in determining the length and cost thereof.

In railroad bridges the fixing of the approach grades sometimes involves the economic solution of the question of a steep grade with pusher engines *versus* an easy grade without them. In this case it is necessary to add to the first cost of the former, the first cost of all the pusher engines needed, plus the capitalized value of the annual cost of their operation and deterioration, and compare the sum with the first cost of the latter. Moreover when figuring the annual cost of operation, it is necessary to include therein the annual expense due to delay of trains caused by stopping, attaching the pushers, and regaining speed.

Geographical Conditions

The layout of a bridge is sometimes influenced to a certain extent by its geographical location, because a structure suitable for the heart of a city might not be appropriate in a country district, and vice versa. Generally the variation involved would be a question of æsthetics, or possibly one of flooring, for sometimes it is necessary to cover over the deck of a railroad bridge so as to permit it to take care also of highway traffic. In mountainous districts where the transportation of large, heavy pieces is either very expensive or altogether impracticable, the layout would be governed by this condition.

Commercial Influences

The principal commercial consideration that will affect the layout of a bridge is the amount and character of the traffic of which it will have to take care. If there is a variety of traffic, such as steam railway, electric railway, wagon, and pedestrian, considerable attention must be paid to the question of how best to take care of all probable combinations of the different kinds. Much money can be saved for a client by a bridge engineer who knows how to handle the question; and much can be wasted by one who is not properly posted on this important subject. An indisputable proof of the correctness of the latter statement is furnished by the notorious case of a proposed bridge to cross the Second Narrows at Vancouver, B. C. In that layout three railway tracks were adopted where two would have served the purpose equally well, with the result that the estimated cost of the structure was increased about seven hundred and fifty thousand dollars, and the project, in consequence, was either killed or relegated for consummation to the dim and distant future.

Property Considerations

Property considerations sometimes have a far greater effect on the layout of a structure than is at all legitimate. For instance, in the case of the Northwestern Elevated Railroad of Chicago, engineered by the author in the early nineties, certain high prices for land caused the company to lay out such a crooked line as to interfere materially with the attainment of a satisfactory train velocity. Refusal of property owners to allow the construction of piers or pedestals on their land will often oblige an engineer to adopt an unduly long span, or even an entirely different type of construction from the ordinary. Again, the necessity for occupying a certain city street will sometimes change entirely the character and layout of an approach to a bridge, and it might affect even the layout of the bridge itself. The method of crossing a railroad track at the entrance to a bridge might alter fundamentally the type of structure, a low bridge with an opening span being adopted if the crossing be at grade, and a high bridge with fixed spans if it be overhead. Public improvements sometimes cause material modifications of plans for proposed bridges; and even projected improvements with prior rights are liable to cause troublesome interference. The author has lately encountered obstructive opposition of this nature on a big bridge project.

General Features of Structure

The question of whether through, deck, or half-through truss spans or girders are adopted is one that will radically affect the layout, but mainly in the line of economics, because deck structures in most cases involve a saving of expense in both substructure and superstructure, in that the

piers are shorter than those for through or half-through spans, and, generally, the spans are narrower, thus causing a saving of metal in both the cross-girders and the lateral bracing. The clear headway required, especially for short spans, is likely to influence the layout more or less.

The possibility of using buried piers and protecting the feet of the embankments near them by rip-rap will not only affect the physical appearance of the bridge, but also it will modify the economics of the crossing.

In case a bridge is to cross a navigable stream, the layout of spans will depend primarily upon whether a swing, bascule, or vertical-lift span is adopted for the opening. If a swing is employed, it will generally require an expensive draw protection, while for a bascule or a vertical lift some comparatively inexpensive dolphins, either with or without cheap fender-walls of sheathed piles, will suffice.

The possibility of building an arch, a cantilever, or a suspension bridge instead of a simple-span structure would affect the layout in many ways—physically, æsthetically, and economically.

Again, the material adopted for construction—whether masonry, concrete, steel, or timber—will have a similar influence.

The matter of shore protection is not likely to affect directly the layout for a bridge, although its use certainly does increase the total cost; but it might be the reason for shifting the crossing to a location where the bank is better protected by nature against scour.

Finally, the layout is affected by the character of the approaches; for they may be of earth embankment, timber or pile trestle, steel viaduct, or reinforced-concrete girders or arches.

Future Enlargement

The possibility of future enlargement of structure ought to receive consideration; and if it be decided that it is at all probable, a study of the layout should be made so as to determine how best to accomplish such enlargement when the time comes for so doing. The points to consider are whether it will be best to build an entirely separate new bridge close alongside, or to put a double-track superstructure on the old, single-track piers by enlarging them or expanding their tops, or, at the outset, to put in large piers and build the superstructure in such a manner that the trusses can be doubled in the future.

Again, it would frequently be good engineering to provide at first only the floor systems necessary to suffice for traffic requirements at the outset, but to design the trusses and substructure so that additional roadways and tracks can be added in the future when needed.

Time Considerations

The time allowed for completing the substructure or the superstructure or the whole bridge may affect the layout, for it is understandable that a

certain type of structure could be built in a certain limited time while another type of structure could not. Again, the length or shortness of the working season that is entirely free from danger of washout of falsework could be a sufficient reason for changing materially the layout—for instance, by necessitating pin-connected spans instead of riveted ones, or steel truss-spans instead of concrete-arch ones, or semi-cantilevering of certain spans instead of falsework erection throughout.

Stream Conditions

The various influences of the stream that is to be crossed are more potent than most other factors in affecting the layout. The high-water- and the low-water-elevations are important features in the designing of the piers; the amount and character of the drift determine the minimum vertical distance between high water and the bottom of the superstructure, and, therefore, aid in settling the pier height; and the amount and consistency of the passing ice constitute an important factor in the design of the piers, especially in respect to their length and the character of their end finish; and any increasing of the cost of the piers tends, for economic reasons, to lengthen the spans.

The clear waterway required to pass the probable maximum flood will often settle the total length of structure; and it may result in raising the high-water mark that was determined in some other manner. The profile of the river-bed and the probable scour of the materials of which it is composed are likely to affect the layout, especially if the piers require expensive protection of mattress work and rip-rap to check the said scour. The frequency and extent of the floods will influence the cost of building the piers—hence also the determination of the layout—as will also the questions of rise and fall of tides, velocities of the passing water, reversal of current, and the existence or possible future building of levees.

A most important factor is the possibility of the permanent shifting of the channel from one side of the river to the other. If this possibility exist, one of three things must be done, viz.: first, two movable spans must be provided; second, some effective method of retaining the channel in one position must be arranged for; or, third, the design must be so made that any fixed span of the structure may at any time be converted into a vertical-lift span.

Foundation Considerations

Important also in the determination of layout are the character and the depth of the substructure foundation. The deeper the piers have to go the longer will be the economic lengths of the spans. Again, the more difficult it is to penetrate the materials overlying the bed-rock or final foundation, the greater the cost of the piers, and the longer the economic spans. The ultimate depths to foundation and the materials to be pene-

trated determine what process of sinking to adopt; and as the cost of the substructure depends upon the said process, so also will the layout.

Navigation Influences

The influences of navigation that are likely to prevail during the time of the contractor's operations may be of such moment as to affect more or less the design and the layout of the structure; although this is not very likely. Again, the possibility in the future of greatly augmented river-traffic may influence the type of movable span adopted.

Construction Facilities

The availability or otherwise at the bridge site of sand, gravel, concrete-stone, a machine shop for repairs, and a reliable source of supplies for the work and workmen, the accessibility or the contrary of the site from the nearest railroad depot or siding, the length and difficulty of wagon-haul or other means of transportation of materials and supplies, the facilities for securing and retaining labor, and the availability of supplies of timber and piling all affect greatly the cost of the substructure and to possibly a somewhat less degree that of the superstructure—hence also the layout of spans and piers.

Erection Considerations

The difficulties that may be anticipated for erection, and the method thereof finally adopted, whether by falsework, cantilevering, semi-cantilevering, or flotation, are important factors affecting the layout of the structure, as are also the questions of the maintenance of traffic and the replacement of an existing bridge.

Æsthetics

Too often the question of æsthetics is totally ignored; but when it is given proper consideration, it may cause modifications in span lengths, truss dimensions, and shapes of piers. How much extra money it is legitimate for a bridge engineer to spend for the purpose of beautifying a structure is a mooted point. It depends greatly upon the designer's appreciation of the beautiful in nature and in art, as well as upon the elasticity of the client's purse and the extent of the influence upon him exerted by his consulting engineer, also upon the location and surroundings. Generally speaking, the best layout for all the other ruling causes is the best also for æsthetic reasons; but there are cases when a little extra expenditure of money, time. and brains will secure great improvement in appearance; and in such cases the beautifying of the construction should, if possible, be accomplished.

MAINTENANCE AND REPAIRS

The cost of maintenance and repairs as well as that of operation may sometimes be a vital consideration affecting the layout of a structure. For instance, when the Jefferson City highway bridge over the Missouri River was about to be built, the bridge company, in spite of the author's forcible remonstrance, let the contract for the structure on the basis of a high bridge with a long and expensive timber trestle approach. Later they were convinced that the annual expense of maintaining the said trestle would be so great as to consume more than the total net income from traffic receipts; hence they had to change to a low bridge design.

THEORETICAL ECONOMICS

From time to time an engineer encounters a bridge problem in which the controlling factor in the layout determination is really that of economics, and then he is happy; for, comparatively speaking, the case is a simple one. A case of this kind occurred in the author's Canadian Northern Pacific Railway bridge across the North Thompson River, near Kamloops, B. C. The structure consists of a number of deck, plate-girder spans, one of which is lifted so as to permit of the passage of small river steamers at certain high stages of water.

The requirements of æsthetics often conflict with those of economics; for it would not look well to let the span lengths change backward and forward, perhaps, to suit the vagaries of an unusual bed-rock profile; hence it is best in many cases to compute the economic span length for average conditions of pier cost and to use one length instead of several. It will generally be found that such an arrangement does not involve any extra expenditure worth mentioning when the cost of structure for that layout is compared with that for the truly economic one. The question of economics, however, cannot be finally settled by adopting simply that structure for which the initial cost is a minimum; because, as pointed out previously, the truly economic bridge is the one for which the sum of the first cost and the capitalized annual cost of operation, maintenance, and repairs is a minimum.

As a conclusion to the general subject under discussion, in order not to discourage young engineers, it might be well to state that any designer who, when determining the layout for any large and important bridge, can and does give full and due consideration to all the factors treated in this chapter, is truly worthy to be termed an expert bridge engineer.

CHAPTER XVI

ECONOMICS OF LOADS AND UNIT STRESSES

THE determination of the live loading for any proposed structure is an economic problem of prime importance; and it often involves the employment of engineering talent of the highest order. As a rule, that loading should be made large enough to take care of the greatest moving loads that may reasonably be expected to come often upon the structure during its lifetime; and as the latter for a well-designed, modern bridge is of indefinitely great length, the problem is by no means an easy one to solve. It must be recognized that the *occasional* application of a load exceeding by 25 per cent, or even more, the one used in making the design, will do no harm to the structure, but that when the excess reaches 50 per cent and is applied often the condition begins to become serious.

While it is right and proper for an engineer to act with precaution by assuming the live load great enough to meet all likely contingencies, it would be uneconomic to attempt to provide for improbable or impossible loadings. Again, while it is feasible to subject a short span to a very heavy loading, it is not so in the case of a long one; and the longer the span the smaller is the chance of its ever having to carry an abnormally great live load. This remark applies more forcibly to highway bridges than to steam-railway structures; although it undoubtedly holds good to a certain extent for the latter, because it is very seldom that long trains are composed entirely of the heaviest kinds of loaded cars. About the only exception to this rule is in the case of a structure on a railroad carrying long trains of fully-loaded ore-cars or coal-cars.

There is one point in connection with highway live loadings that is of importance, viz., that all new highway bridges should be proportioned to carry heavy trucks. The concentrated loads caused by these are so great that the old-fashioned wooden-joist floors will not be able to carry them. There is no telling how far from home these exceedingly heavy auto-trucks will travel, so that no out-of-the-way highway bridge will be safe from their invasion. Again, it must be remembered that auto-truck loads are rapidly on the increase; hence one should be liberal in his assumptions for this feature of the live loading.

In respect to live loading in general, it seems almost unnecessary to mention that the amount thereof decreases slowly with the span-length. In steam-railway structures this is primarily because the effect on the total span-length of the greater weight per foot of the coupled standard locomo-

tives, as compared with that assumed for the loaded cars, diminishes as the span-length increases; but in highway bridges the lessening is due to an application of the theory of probabilities. For the electric-railway loading the diminution in the equivalent live load per lineal foot exists only when the length of span is greater than the assumed total length of train, and the greater the difference in length the greater the said diminution of loading per lineal foot of span. On pages 110 to 116, inclusive, of "Bridge Engineering" are given curves of equivalent uniform loads for trains of two, three, four, five, and six electric-railway cars and for all the classes of loading from 15 to 40. These are carried out to a length of only 600 feet; and in case of longer spans it would become necessary to lengthen them either accurately by computation or approximately by visual extension.

For ordinary railroad bridges the determination of the proper live load to adopt is generally a simple affair, because the company is likely to have a standard of its own; but sometimes it may be advisable to depart from this, in order to provide for some unusual condition or complication. For highway bridges, though, it is a different matter, because the amount and character of the traffic will depend greatly on local conditions; and, consequently, the size of the loading is a question of judgment. In exercising it, one should look to the possibility of a material increase in the weights to be transported; and should be governed accordingly.

The standard steam-railway loadings given in Chapter VI of "Bridge Engineering" are still sufficient for maximum requirements, and it is not likely that the greatest (Class 70) will ever be materially exceeded, unless some fundamental improvement be made in the character of railway roadbeds. The distribution of weight on the axles of the locomotives is gradually being changed, but the weight thereof per lineal foot of track does not appear to be augmenting to any great extent.

The heaviest live loads for electric railways given in the aforesaid chapter have not yet been reached, except in the case of electrified steam-railroads; and these are not treated as real electric railways, as far as bridge loadings are concerned. Ordinarily, Class 25 is heavy enough for the street-railway live-loads on highway bridges; but sometimes it is advisable to proportion for Class 30, in order to provide for future heavy suburban cars.

Classes A, B, and C for highway loadings are sufficiently heavy, even if it be possible to crowd vehicles and pedestrians so close together as to cause the actual loadings to exceed them; because, when much crowding occurs, the speed of travel is materially reduced, and then the effect of impact may practically be ignored. Class A loading is so great that, for comparatively long spans and wide decks, it may be used to cover the average loading over the full width of deck from electric-cars, vehicles, and pedestrians—and this applies in nearly all cases to suspension bridges. But in that type of bridge it must be remembered that for the stiffening trusses the equivalent uniform live load per lineal foot of structure must

be taken for about four-tenths of the span-length and not for the full length, as is the case for simple spans.

In cantilever bridges judgment must be used in determining the lengths for equivalent uniform loads for the different portions of structure. For instance, in Type-A cantilever, shown in Fig. 12*a*, for the suspended span its total length is to be used; for the cantilever arms the sum of the length of the suspended span and that of the portion of the cantilever arm that is loaded for maximum stress on the member considered; and for the anchor arms the total length of the loaded portion of one such arm for direct loading and that of one cantilever arm plus suspended span for reverse loading. A skilful bridge designer can often legitimately economize in his work by making a careful study of the live-load question and cutting down the loading as hereinbefore indicated.

Similarly a study of the question of impact allowance and the utilization of the very latest information thereon will result in a certain amount of economy. For instance, since "Bridge Engineering" was written, the author has found it advisable to modify the impact allowances recommended in Chapter VII thereof in the following particulars:

First. For electric-railway loads use the same impact as for highway loads. This modification is based upon some late experiments by Prof. Turneaure.

Second. When a reinforced-concrete base is used for the pavement of a highway bridge with similar slabs for the footwalks, the impacts given may be reduced twenty-five (25) per cent.

Third. For reinforced-concrete arches and girders, without earth filling, the impacts may be reduced fifty (50) per cent.

Fourth. For earth-filled, concrete arches, they may be reduced seventy-five (75) per cent.

The determination of the width of deck that it is best to employ is an economic question of salient importance. A narrow roadway tends to induce slow travel, owing to the necessity for reducing the speed of automobiles in narrow places so as to avoid collision. For two lines of travel at high speed a width of twenty-two feet is advisable, although one of twenty feet is often considered sufficient because it is practicable to pass most auto-vehicles on a width that is two or three feet less. There are many bridges in country districts that are only eighteen feet wide and some as narrow as sixteen feet; but these do not meet with the approval of automobile drivers, because a load of hay will almost block the structure—besides, when one is driving at high speed, he does not like to be compelled to slow down when he is approaching a bridge upon which there is another vehicle.

A thirty or thirty-two-foot width of main roadway is better than one of twenty or twenty-two feet, in that it will permit a fast vehicle to turn out and pass a slowly-moving one; but there is always the danger of running into an automobile of the other line that is trying to accomplish the same purpose. A roadway of forty or, preferably, forty-two or even forty-

four feet in the clear between curbs will take care nicely of four lines of automobiles, and the middle twenty feet thereof may carry a double track for an electric railway.

From the viewpoints of both economy and public convenience it is better to pass at once from a two-line-travel deck to a four-line-travel deck, because a width great enough for only three lines is unsatisfactory in that, as before indicated, it offers to reckless drivers a fine opportunity for head-on collision—and, again, it is not as suitable for high speed as a wider structure. It is true that in the case of a total break-down it has a decided advantage over the two-line-travel deck, permitting vehicles to pass around the obstruction, but that is not sufficient reason to warrant its adoption.

Footwalks are often made much wider than necessary, but sometimes too narrow. The minimum width should be five (5) feet, which will just permit two people to walk abreast and to carry umbrellas. Any smaller width would be uncomfortable, and one a foot wider would be better. A width of eight (8) feet is generally sufficient for all requirements, but sometimes ten (10) or even twelve (12) feet are called for. If travel in both directions is permittted on the same footwalk, it should not be made less than seven (7) feet wide, and preferably nine (9) or ten (10), but it is seldom good policy to allow such traffic, because the pedestrian travel will be faster if it is always kept to the right-hand side of the structure.

If a bridge is very long, it is not likely that there will be much pedestrian travel over it, unless there be some special inducement such as beautiful scenery or cool breezes in hot weather, consequently the omission of sidewalks, either entirely or temporarily, becomes an economic question of some importance. Sometimes, however, the obtaining of a charter for building the bridge is conditioned upon putting on sidewalks; and in that case the only economy practicable would be to obtain from the authorities permission to provide for their future addition and to omit them temporarily until the development of traffic indicates their necessity. In very long bridges it is economical to reduce the sidewalk width to the absolute minimum of five (5) feet, owing to the small number of people that are likely to pass over the structure on foot.

When a steam-railway bridge has to carry electric-railway cars as well, it is often economic to adopt the expedient of gauntleted tracks, i.e., separate lines of rails for the electric railway lying quite close to the steam-railway rails and with no connection thereto, switches being omitted and the steam-railway rails being cut so as to permit the wheels of the electric-railway cars to cross them on a frog. This expedient obviates the necessity for increasing the railway live loads in order to provide for the electric-railway traffic.

The question of what arrangement of decks to adopt for the accommodation of several kinds of traffic is an economic problem that sometimes arises in a bridge engineer's practice. It has lately come up in the author's

in a case where a double-track, steam-railway bridge had to carry in addition a double-track electric-railway, wagon-ways, and footwalks. The choice lay between a double-deck structure with the steam railways below, a combined double-track electric-railway and wagon-way above between trusses, and footwalks outside of the wagon-way; and a single-deck structure with the double-track steam-railway between trusses as before, a combined single-track electric-railway and wagon-way on cantilever brackets outside of each truss, and a footwalk outside of each wagon-way. For reasons unnecessary to state, the gauntleted tracks between trusses for the electric railway were barred. The comparison was hardly a fair one, because the single-deck layout provided facilities superior to those afforded by the other layout in relation to rapid transit,—there being two roadways of twenty-two feet each instead of one roadway of thirty feet—besides avoiding a climb of some twenty-five feet. While the cost of the main spans was greater for the single-deck structure than for the double-deck one, on the other hand there was a saving in the lengths and costs of the approaches. It is impracticable to make any general statement of comparative costs or advantages of these two types; and each case as it arises, will have to be worked out by itself as a special economic problem. Moreover, one layout or the other, irrespective of cost, will be preferable from the point of view of service or accommodation, hence this feature will have to be given serious consideration.

As a rule, bridges for carrying both railway and highway traffic are located in or near large cities, although an occasional structure of this kind is found in country districts. The principal advantage of this type of bridge is the saving in first cost, and its principal disadvantage is a reluctance to cross over it on the part of timid drivers, whose horses may be frightened by the trains. The saving in first cost of a combined railway and highway bridge, as compared with two separate bridges for railway and highway traffic, is considerable, because the piers for the combined bridge are but little, if any, more expensive than those for the railway bridge, and because the extra metal for the super-structure of the former in comparison with that of the latter is very much less in weight than the metal required for a separate highway bridge. The prejudice against combined bridges on account of danger is almost wholly unfounded, for horses soon become accustomed to railway trains, and, when screens are employed to hide the latter, but little trouble is experienced on account of frightened animals. These screens may be made either slatted or solid, the former offering less resistance to the wind, and the latter being the cheaper. Moreover, automobiles have almost entirely displaced horses in highway traffic.

The advent of the electric railway has somewhat complicated the question of designing combined bridges, for now it is often necessary to accommodate all kinds of traffic on the same structure.

Combined bridges may be divided into the following classes:

1. Structures having a single deck for all kinds of traffic, the railway

occupying the center of the bridge, and the electric railway lying close to one truss.

2. Structures having a single-track railway at the middle, a narrow footwalk on each side thereof inside of the trusses, and cantilever brackets outside of the latter to carry roadways and electric lines. This arrangement may be varied by running the electric cars over the main railway track, thus leaving the wings free for vehicular traffic.

3. Structures having a double-track railway inside of the trusses, with long cantilever brackets outside carrying wagons and electric lines next to the trusses and pedestrians outside. This arrangement may be varied, as in Case 2, by carrying the electric trains on either one or both of the main railway tracks.

4. Structures having a double-track railway inside of the trusses, with short, cantilever brackets for wagon and electric-railway traffic outside, and either a single passageway overhead at the middle for pedestrians, or two passageways therefor on overhead brackets outside of the trusses. As before, this arrangement may be modified by running the electric trains over the main railway tracks.

5. Double-deck, single-track structures carrying a railway train on one deck and vehicles and pedestrians on the other. If electric cars also are carried, they should generally use the railway track on account of the narrowness of the bridge; but by putting the railway below and using cantilever brackets above, the electric cars may share the wagon-way and run over either one or two tracks. When the electric cars and the vehicles occupy jointly the upper deck, it is generally best to carry the pedestrians by cantilever brackets on the lower deck, as the structure might be too narrow to warrant caring for them above by footwalks outside of the joint wagon and electric car roadway and because permitting them to use the said joint roadway would be too hazardous.

6. Double-deck, double-track structures carrying railway trains on one deck and vehicles, electric trains, and pedestrians on the other, or with the electric trains using the steam railway tracks. The vehicles and electric trains may either occupy the same roadway, or the former may be carried on cantilever brackets, leaving the middle portion of the deck for the latter. In such a bridge the footwalks should be on cantilever brackets, either above or below, outside of the other roadways.

In double-deck structures where the steam railroad is below, it is necessary to use every precaution for keeping the locomotive fumes away from the upper deck, as smoke rising through the floor frightens horses even more than does the train itself. Moreover, smoke is exceedingly disagreeable to everybody passing over the structure. Again, the question of protecting the highway floor from being set on fire by sparks from locomotives must be satisfactorily solved in this combination.

Class No. 1 is the cheapest possible kind of combined bridge, and at the same time the most unsatisfactory, for when a railroad train is about

to pass over the structure all vehicular and electric-railway travel must be kept off, and because pedestrians must look out sharply for their safety when on the deck with a railway train crossing. Their danger is really greater, though, when an electric train is passing a team or teams. The least allowable clear width of bridge for this class of structure is twenty feet, the electric cars running on a third rail and on one of the rails of the main railway.

Class No. 2 is a very satisfactory type of structure. The author has designed and built several bridges of this kind, the largest of which is the Combination Bridge Company's bridge over the Missouri River at Sioux City, Iowa. It consists of two draw-spans of 470 feet each and two fixed spans of 500 feet each, besides the deck approach spans, the distance between central planes of trusses being twenty-five (25) feet.

Class No. 3 is also a satisfactory type of structure. The author once built a large bridge of this type, viz., the one across the Missouri River at East Omaha, Nebraska. This class of structure involves very heavy metal-work; but it is not uneconomical.

Class No. 4 is an unusual type, and is not likely to be called for very often, although the author has had occasion to figure on bridges of this kind.

Class No. 5 gives a satisfactory distribution of traffic, as was proved by the author's bridge over the Fraser River at New Westminster, British Columbia. In this the steam railway and the electric cars occupy a single track on the lower deck; and vehicles and pedestrians use in common a sixteen (16)-foot clear roadway on the upper deck.

Early in 1908 in preparing a design for a combined bridge to carry a railway, a street-railway, wagons, and pedestrians over the Second Narrows of Burrard Inlet at Vancouver, British Columbia, the author evolved a rather novel method of dividing the traffic. The bridge was to be built at first to carry only the railway and the street-railway, but provision was to be made to take care of wagon and pedestrian traffic in the future. The distance between central planes of trusses being restricted from motives of economy to the least consistent with the Dominion Government's requirements for clear roadway—in this case nineteen (19) feet—it would have been improper construction to put twelve (12) foot roadways outside of the trusses and six (6) foot sidewalks outside of these; for such an arrangement would make each cantilevered portion of the deck wider than the distance between trusses, while good practice does not permit it to exceed two-thirds thereof. As the clearance above high water was ample on account of there being an overhead crossing of the Canadian Pacific Railway tracks at the south end of the structure, it was suggested to suspend the footwalks from the cantilever brackets that carry the roadways. This would necessitate small roofs to protect pedestrians from the roadway drippings. The arrangement described was shown by cost estimates to be exceedingly economical, but it was objected to on account of its interfering with the running of certain small craft under the swing span.

Class No. 6 represents a very good arrangement which can be modified to suit nearly any conditions of combined traffic. A good example of this type is the author's bridge over the Missouri River at Kansas City, Mo., owned by the Union Depot, Bridge, and Terminal Railroad Company, and known as the Fratt Bridge.

In designing combined bridges of all classes except No. 1, a considerable economy of metal may be effected legitimately by keeping the total live load for trusses as low as is proper with reference to the theory of probabilities. For instance, in Class No. 2 or Class No. 5 the live load for trusses may be determined by adding to the equivalent uniform live load for the steam-railway tracks, given by the diagram in Fig. 6*e* of "Bridge Engineering," a much lighter highway floor load per lineal foot of span than that prescribed in the specifications; because when the greatest train load is on the bridge, the chance of having simultaneously a heavy highway live load is very small. The longer the span the smaller may the live load per square foot of floor be taken when finding the total live load for the trusses. Again, in Classes No. 3 and No. 4 it would be legitimate to take the truss live load per lineal foot for the railway equal to twice the car load per lineal foot, and add thereto a small highway live load as in the last case. Finally, in Class No. 6 in case of a four-track bridge with cantilevered highways and footwalks, it would be proper to assume the live load for the trusses equal to the sum of the car loads per lineal foot on the four tracks and ignore entirely the vehicular and pedestrian loadings; for the greatest probable live load from all classes of loading would never exceed the said four car loads.

This reduction of live load, however, can readily be carried to extremes, as was the case in the first accepted design of the great cantilever bridge over the St. Lawrence River near Quebec, and as is likely to be the case whenever the preparation of the specifications for a bridge is left either directly or indirectly to the contractor who is to build the structure. Good judgment, uninfluenced in any way by considerations of personal gain or by motives of false economy, should rule in the establishment of the live loads for the trusses of "combined" bridges.

It is very seldom that the designing engineer of a bridge has an opportunity to economize by the manipulation of unit stresses, because ordinarily he employes standard bridge specifications; and truly it is better for him to do so, for the reason that this question has already been very thoroughly threshed out; but, as will be explained presently, in certain structures, such as trestles, he may have an opportunity to employ his judgment in the determination of unit stresses for certain unusual but possible combinations of loads.

In modern specifications the aim is to have a fixed unit stress for each kind of material for all but extraordinary combinations of loads. In metals it is generally proper to place the ordinary intensity of working stress at one-half of the elastic limit of the material; but with high-alloy steels it

should not exceed one-third of the ultimate strength. This restriction applies mainly to heat-treated eye-bars.

Some engineers deem it good practice on the score of economy to use high-carbon steel for reinforcing-bars; but the author is opposed to this for two reasons; first, such metal is liable to crack when being bent cold in the field; and, second, a high intensity of working stress on the bars tends to permit the formation of cracks in the concrete in their vicinity.

Very infrequent loads will permit overstresses, or, in other words, encroachment on the factor of safety. A future increase of live loads will affect differently different portions of a bridge. This is sometimes allowed for by reducing the unit stresses on certain members; but it is better to design the structure so that an increase of loading of fifty per cent will not overstress any member more than fifty per cent.

Some designers specify abnormally heavy loadings and correspondingly greater unit stresses. This gives well balanced structures; but the method is illogical, and it has a tendency to deceive. If such a practice were to become established, there would be a liability on the part of inexperienced engineers to employ high unit stresses with normal loads, which would be dangerous or at least ultimately uneconomic.

In very-long-span bridges the selection of live loads and unit stresses is of extreme importance; because a small legitimate increase in unit stresses or a small reduction of loading may result in a comparatively large saving in cost, due to decrease of dead load.

If, for the sake of either a real or an imaginary economy, an increase in unit stresses must be adopted, it is better to make it frankly in the main members, where the danger from overstress is least; and then the fact of the existence of this condition will be apparent. But if skimping were done in the detailing, there might easily be developed weaknesses of such a serious nature as eventually to cause disaster. Again, more money can be saved by reducing the sectional areas of main members than by trimming the details.

In the designing of ordinary truss bridges, the computer has practically no choice as to how the various stresses which he figures are to be combined, because the standard specifications by which he is governed indicate the method very clearly; but in structures unusual as to either type or magnitude and in trestles the designer should have some option in making the combinations. No hard-and-fast rule can well be given to cover all cases of spans of unusual size and character; except to state that the engineer should employ his best judgment in relation to possibilities and probabilities, stressing the standard amount for combinations that are likely to occur and increasing the intensities of working stresses as the combinations considered become more and more improbable of realization, up to the limit of an excess of fifty (50) per cent.

In bridges proper, with the exception of arches having less than three hinges, the only unusual combination is that of the ordinary stresses and the

wind stresses, to allow for which standard bridge specifications permit an increase of thirty (30) per cent over the regular intensities of working stresses; but in trestles there may be combinations of live-load, impact, dead-load, centrifugal-load, wind-load, traction-load, and temperature stresses; hence the computing of some of the sections for these structures is a complicated matter.

As stated in the specifications for designing given in Chapter LXXVIII of "Bridge Engineering," the columns of steel trestles are to be proportioned thus:

First. For live load, impact, centrifugal load, and dead load, with the usual intensities.

Second. For live load, impact, centrifugal load, dead load, and wind load or traction load, with an excess of thirty (30) per cent over the usual intensities.

Third. For live load, impact, centrifugal load, dead load, wind load or traction load, and temperature, with an excess of forty (40) per cent over the usual intensities.

Fourth. For live load, impact, centrifugal load, dead load, traction load, and wind load, with an excess of forty (40) per cent over the usual intensities.

Fifth. For live load, impact, centrifugal load, dead load, traction load, wind load, and temperature, with an excess of fifty (50) per cent over the usual intensities.

The preceding combinations and excess percentages of intensities were adjusted after much deliberation; and their publication in "Bridge Engineering" was the first complete exposition of the matter ever made in print. In the preparation of specifications theretofore, the question had been deemed too complicated for written treatment and had been left for settlement entirely to the judgment of each individual designer. A study of the preceding adjustment will show that the greater the improbability of any combination the greater the intensity of the working stress adopted. The worst combination (which, really, never could occur) would stress the metal up to three-quarters of its elastic-limit, which is perfectly safe for an occasional loading. It is much better to take into account all possible combinations and to stress the metal high for the worst summation than to ignore such combinations entirely and trust to luck that they will never occur, as is too generally done in trestle designing. On the other hand, though, it would be extravagant practice to combine all the possible stresses and use either the ordinary intensities or even these increased by the usual thirty per cent allowance for the inclusion of wind. Trestle proportioning hitherto has been rather unscientific, and it is to be hoped that it will soon be improved. When all is said and done, however, it is impracticable to eliminate entirely individual judgment in the designing of high steel trestles, because in some cases local considerations will permit of the reduction or even the ignoring of certain stresses. For instance,

when a trestle is situated near the middle of a sharp curve or near the apex of two heavy, rising grades, it would be bad judgment to assume a high velocity of train when finding the stresses due to centrifugal loading.

The combination of stresses in cantilever bridges and in arches is not so complicated as it is in trestles; but it is to be noted that the sections of members do not need to be increased because of erection stresses, unless such total stresses (including those from wind under an assumed probable pressure of ten (10) or fifteen (15) pounds per square foot) raise the intensities on the computed sections above those specified for a combination of the usual loads with wind.

In summing up stresses care must be taken to add only those that can act simultaneously, because some stresses can never occur together; for instance, live load and erection stresses in cantilevers and in arches erected by cantilevering, and live load and wind stresses in highway bridges. This word of warning seems almost unnecessary; nevertheless a careless computer is liable to sum up stresses that cannot act together, as the author knows from personal experience.

At the present time there is a division of opinion among bridge specialists concerning the combination of stresses of opposite kinds, some of them going so far as to ignore altogether the effect of reversing stresses. Until good reason is offered for making a change, the author intends to adhere to the method which he advocates in "Bridge Engineering." It is as follows: If the cause of the reversal be wind, the effect of reversion is ignored, because not only is there generally a long interval between reversals, but also the maximum wind stress on any piece is of infrequent occurrence. Reversals due to live loads combined with impact are divided into two classes; first, those which occur in succession during the passage of a live load over the structure, and, second, those which are caused by different loadings. In the first case, each of the two kinds of stress is to be increased by seventy-five (75) per cent of the other, then the section required for each combination is to be computed and the larger of the two adopted. In the second case the procedure is similar to that just described, except that the percentage to be added is fifty (50) instead of seventy-five (75). The author does not deny that it might be perfectly safe to reduce these percentages to fifty (50) and twenty-five (25), respectively, but he is decidedly averse to ignoring altogether the effect of reversion.

In any case it would require some exceedingly strong evidence to induce him to change his method of computing the number of rivets for connecting main members, viz., to add together without any reduction the two stresses of opposite kinds and proportion for the sum. Stress reversal is certainly harder upon the connecting rivets than upon the members themselves which they join.

After all, this controversy about the proper allowance for reversal of stress may amount to much ado about nothing, because, as pointed out in Chapter XI, the difference in the weights of metal in two continuous-truss

spans of 775 feet each, when for all reversals an addition of seventy-five (75) per cent of each stress to the other stress was made, and when the effect of reversion was entirely ignored, was only two and a half ($2\frac{1}{2}$) per cent. Of this one and a half ($1\frac{1}{2}$) per cent were due directly to the increase in the sections of the pieces in which reversion occurred, and one (1) per cent to the augmented dead load. Such being the case, and in view of our limited knowledge concerning the wearing effects of stress reversal, it seems hardly worth while at present to make any change in the practice prescribed in "Bridge Engineering." On the other hand, though. to be perfectly fair, it should be pointed out that reversal of stress may have a greater effect on some other kinds of bridges, such as arches, than it does on continuous-truss structures.

Until the appearance of the January, 1920, issue of the Bulletin of the American Railway Engineering Association, no writer of bridge specifications, as far as the author knows, had ever attempted to specify intensities of working stresses for combinations of live and dead load stresses with secondary stresses. In the Report of "Committee XV—on Iron and Steel Structures" (of which, by the way, the author is a member), published in that issue, there appears the following clause, numbered 47:

Secondary Stresses:

Designing and detailing shall be done so as to avoid secondary stresses as far as possible. In ordinary trusses without subpaneling, no account usually need be taken of the secondary stresses in any member whose width, measured in the plane of the truss, is less than one-tenth of its length. Where this ratio is exceeded, or where subpaneling is used, secondary stresses due to deflection of the truss shall be computed. The unit stresses specified in Article 38 may be increased one-third for a combination of the secondary stresses with the axial stresses.

This allowance of thirty-three and a third per cent applies only to combinations of live load, centrifugal load, impact, and dead load with secondary stresses; but if other loadings, such as wind loads, traction loads, and temperature effects, are added to the combination, the increment of intensity ought to be increased, according to the designer's judgment, up to a limit of fifty per cent.

There is an important question now before the engineering profession for settlement, which, strictly speaking, is one of economics, viz.: the proper relation between intensities of working stresses for bridge members in tension and compression. In the author's opinion, there is no valid reason for the drastic cut in compression intensities made of late in bridge specifications by several of the leading technical societies. Concerning this question he wrote for the Jan. 16, 1919, issue of *Engineering News Record* the following communication:

The report of the Committee on Column Tests of the American Society of Civil Engineers appears to have caused some fright among bridge designers; for I notice that the Engineering Institute of Canada has lately published a "General Specification for Steel Railway Bridges" in which the unit stress for columns is given by the formula,

$$p = 12{,}000 - 0.3\ (l/r)^2,$$

in which l = the unsupported length of the column in inches and r = its least radius of gyration in inches.

This is quite a sudden jump from the old standard formula,

$$p = 16,000 - 60\ l/r$$

The change indicates one of two things: Either that hitherto we have been overstressing compression members 15 to 25 per cent, or that there is going to be wasted a vast quantity of metal in the future.

Moreover, in making such a sweeping change in the compression formula, the writers of the Canadian specifications were not consistent, because in Clause 47 they allow for the compression flanges of beams an intensity of

$$16,000 - 200\ l/b.$$

In the case of railway stringers l/b is generally in the neighborhood of 10; hence the intensity of working stress is about 14,000 lbs. As b is equal to about 4.5 r, for $l/b = 10$ we shall have $l/r = 45$. Substituting this in the Canadian column formula gives

$$p = 12,000 - 0.3\ (45)^2 = 11,400 \text{ lbs.}$$

In the case of one compression member of a bridge, it appears to be legitimate to stress the metal up to 14,000 lbs. per square inch, and in another up to only 11,400 lbs. per square inch, a difference of 23 per cent. And there is no valid reason for stressing differently a strut which forms a part of the top chord of a truss and a strut which forms a part of the top flange of a beam. "Consistency, thou art a jewel!"

If the correctness of the formula for compression flanges of beams be conceded—a formula which, for a dozen years or more, has been one of the clauses of the American Railway Engineering Association's standard bridge-specifications, and which the Engineering Institute of Canada has appropriated without change—and if it be granted that b is generally equal to about 4.5 r—why should not the equivalent formula,

$$p = 16,000 - 45\ l/r,$$

apply in general to struts with fixed ends? Be it noticed that this formula gives higher results than does my old formula,

$$p = 16,000 - 60\ l/r.$$

The American Railway Engineering Association is getting ready to trim down materially its old intensities for steel struts, although not to the extent that the Engineering Institute of Canada has done.*

What aggravates the effect of this proposed decrease of compression intensities is that, simultaneously therewith, the American Railway Engineering Association is contemplating increasing its tensile intensity of working stress from 16,000 lbs. to 18,000 lbs ; and other specification writers are advising that it be made as high as 20,000 lbs.

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* The 1920 A. R. E. A. bridge specifications permit the following intensities of working compressive stresses:

Struts . . . . 15,000 $- 50\ \dfrac{l}{r}$ but not to exceed 12,500 lbs.

Flanges of girders. . . . . . 14,000 $- 200\ \dfrac{l}{b}$.

The previous strut formula of the A. R. E. A. was 16,000 $- 70\ \dfrac{l}{r}$; hence the new formula gives lower results when $\dfrac{l}{r}$ is less than 50 and higher results when it is greater.

To an outsider it must look as if the bridge engineers of this country were losing their heads!

The compression tests that started the present wave of apprehension did not have governing conditions corresponding properly to those of actual truss-members; hence I would suggest that, before the bridge engineers of America take the drastic step of assuming steel in compression to be only 60 or 70% as strong as the same material in tension, some really practical tests of struts be made under conditions corresponding to those in actual structures. Such a series of tests would cost considerable money; but, if the Bureau of Standards at Washington were to indorse the suggestion to make them, it ought not to be at all difficult to obtain from Congress an ample appropriation for the purpose.

The method of conducting these tests that I have in mind is this:

Let there be built a five-panel, riveted-truss bridge of about 100-ft. span; let the middle panel-lengths of the top chords thereof be made decidedly weaker than all the other portions of the structure; and let a uniformly-distributed live-load be applied at the panel-points by hydraulic pistons. All portions of the structure, including the two weak members, should be scientifically detailed, so that failure will inevitably take place in the main portions of the weak struts and not in the details thereof, and so that the bridge can be used for a long series of tests to destruction of the said mid-panel lengths of the top chords. The weak members should be attached to the connecting plates with an ample number of rivets to develop the full strength of the test pieces; and these rivets should be removed carefully after each test to destruction is completed.

Of course it would be entirely practicable to vary the value of l/r in the different tests, provided that the attachment of the test-strut be not made eccentric.

There should be a supporting platform beneath the span to prevent its falling any material distance when failure occurs.

By adopting a weak vertical post instead of a weak panel-length of top chord, and by loading (at the elevation of the latter) three panel-points only, a series of tests could be made on vertical posts. A similar series could be carried out on the inclined end posts.

A cheaper method than that just described, but not quite as satisfactory, would be to build a single truss, instead of the complete span, and steady it laterally but not vertically, and then to apply the test loadings directly above the top chord at the panel-points. A great advantage of the span tests as compared with the truss tests is that duplicate tests could be made simultaneously on similar members. Moreover, the span tests would be in practically exact accord with actual conditions of loading, while the truss tests would not.

By removing occasionally the test loading, the elastic limits of the struts could be ascertained through noting the absence or otherwise of permanent set.

Experiments similar to those described could be made on a pin-connected span or a pin-connected truss so as to determine the strength of struts with hinged ends.

In view of the immense amount of bridge manufacture and construction that is likely to be done in the United States within the next five or ten years, the making of this proposed series of tests is worthy of being considered a matter of national importance.

In order to elaborate the preceding communication, the author had his associate engineer, F. H. Frankland, Member American Society Civil Engineers, write another letter to *Engineering News-Record* embodying certain subsequent thoughts upon the details of the proposed series of experiments, which letter was published in the issue of March 6, 1919. It reads as follows:

With reference to the matter of formulæ for steel columns, dealt with in recent articles and letters appearing in *Engineering News-Record*, the writer wishes to take this opportunity of drawing attention to the fact that existing column formulæ are not founded on really proper tests, and that, therefore, engineers should not rest satisfied until such proper tests have been made, and, from the empirical rules deduced therefrom, a really sound column formula has been evolved.

No column experiments have yet been made which conform to actual conditions existing in modern bridges, as very few columns nowadays have hinged ends, and flat-end tests are likely to be very deceptive, because of corner bearing.

The opinion expressed by Mr. Charles E. Fowler in his letter on page 343 of your issue of Feb. 13, 1919, to the effect that a straight-line formula is to be desired on account of its simplicity, is to be commended, as there is every probability that a satisfactory straight-line formula can be derived from proper tests. It is to be noted that some of the formulæ given by Mr. Fowler in his letter use the $(l/r)^2$, this being a holdover from the old Euler formula, which was established for greater values of l/r than are used in practice.

With reference to Dr. Waddell's scheme for testing top-chord members by means of experiments conducted on a specially-built, full-size bridge, the writer begs to make the following suggestions for a practical method for making such tests: Build a single-track, Class 70, through-truss bridge, with the weak member designed for, say, Class 40 loading, so that the destruction of the weak member could be accomplished without injury to the rest of the structure, thus facilitating repairs for repeated tests. The connection plates for the weak members should be extra strong. After the compression tests are completed, a series of corresponding tests on tension members should be made, so as to determine the real efficiencies of both tension and compression members and their comparative strengths. Unless this kind of a test be carried out, we shall never have any really reliable test data, as the existing data are, to a great extent, misleading. It is advisable that more than one value of l/r should be tested for.

It should not be overlooked, in making up a design for a test structure, that there is a necessity for thorough detailing throughout, especially in connection with the weak members, so that failure will be in the form of a square break instead of being due to any insufficient detail, thus removing ambiguity. It would be advisable to build the permanent members of the trusses of the test bridge of nickel steel, so that the permanent and the weak sections would be somewhat alike, if the weak section were of the ordinary structural grade of carbon steel.

The most satisfactory method of applying the load would be by means of hydraulic presses or jacks, to be applied at panel points, and all connected so that the load would be uniform. Pressure records should be automatically made. Control of the applied load by this means would be very simple and positive. It would be desirable to make tests for low values of l/r, so as to establish the necessity or otherwise for limiting intensities of working stress.

Many of the tests we are relying on at the present time for our formulæ were made on badly designed struts; and I venture to say that tests on properly designed struts would show up stronger than the present tendency of the profession in strut design indicates.

If the series of tests above suggested were thoroughly carried out, the engineering profession would possess some authentic knowledge concerning the actual strengths of both tension and compression members in bridges, and a large amount of guess work would be avoided in future constructions.

As yet no steps have been taken to materialize this suggestion of the author's. Probably the profession will say that it is up to him to see that his proposed tests are made; and, possibly, after this treatise is completed and launched, he will find time to make an attempt to induce the

Federal Government to undertake the said tests through the Bureau of Standards.

The matter of allowing higher intensities of working stresses for old bridges in service than those specified for the designing of new structures is treated at length in Chapter XLI on "Economics of Maintenance and Repairs."

There is an economic question concerning loads to which but little attention has hitherto been paid, viz., the best way to compute the total loading for a foundation pile. Most engineers ignore impact altogether in estimating the load on piles supporting piers of bridges and trestles, and the author has often done so; but there are cases in which such practice might be unsafe. An instance of this kind occurred in a competitive study made by the author in relation to the rebuilding of the Galveston Causeway after a large portion of it had been destroyed during a hurricane accompanied by a tidal wave. There the piles were comparatively short; and they passed through a thick layer of very soft material before reaching a somewhat firmer one. The layout under consideration was one of reinforced-concrete girders; hence the spans had to be short and the piers small. Under such conditions the vibration from passing trains certainly would have reached the piles from the spans with comparatively little diminution in effectiveness; hence it was essential to allow for impact on the said piles. They had to be proportioned also for effect of thrust of braked trains; but, as the thrust would have been exerted when the train speed was slowing down, it would not have been logical to combine the thrust-effect with the full value of impact. That was an instance where the consulting engineer's judgment had to be relied upon to determine the proper combination of loads, and where a familiarity with the principles of true economics would be of great value to the owners.

The reason why the impact on foundation piles may either be assumed comparatively small, or possibly ignored altogether, are as follows:

First. If allowed for at all, the impact should be assumed for a span-length equal to the sum of the lengths of the two spans which the pier under consideration helps to support.

Second. The impacts given by formula for any span are the greatest that can come upon any main truss-member thereof, and are much larger than those for the span as a whole, as indicated by the ratio of mid-span deflections under the same load when moving and when quiescent.

Third. The critical speed which produces the impact given by formula is likely to be developed very seldom, if at all, on any particular bridge.

Fourth. The massiveness of the pier will absorb some of the shock that reaches its top before the said shock passes to the base.

Fifth. As the tops of the piles are encased in the mass of concrete, they will act together as a unit and thus lessen somewhat the impact per pile.

CHAPTER XVII

ECONOMICS OF TIME AND MONEY IN MAKING COST-ESTIMATES FOR BRIDGES

ONE of the author's fundamental motives in writing "Bridge Engineering" was to provide bridge specialists with a means of estimating quite accurately, and at the same time very quickly, the costs of substructure, superstructure, and approaches for any kind of bridge upon which they are ever likely to have to figure. Rather to his surprise, the numerous reviewers of the work almost entirely overlooked this important characteristic of the treatise. As it is one of the most valuable features thereof, the author, in order to make known this special usefulness, drafted a set of four problems for solution, solely by means of the curves of quantities, tables, formulæ, and other information scattered throughout the book; and arranged for a series of competitions among the senior students in a number of technical schools, and afterwards, by a comparison of the marks resulting from a pre-arranged system of grading, effecting a competition among the said schools themselves by comparing in the columns of the technical press the sums of the marks of the three prize winners in each school. The judges were appointed, and the system of marking was fixed, all ready for the comparison of the unsigned competitive papers; but the entering of America into the Great War so upset the technical students all over the country that the competition had to be abandoned. The author is ready to provide the promised prizes (books of no great financial worth, but valuable as souvenirs of success in competition), in case that any of the teachers of engineering show a desire to have the offer renewed for a new set of problems. As a matter of possible interest, the original list of questions is reproduced at the end of this chapter; and the author suggests that, for the sake of practice in quick computation, engineering students try their wits on the solution of them.

In the author's practice during the last three or four years he has made very quickly many cost estimates on steam-railway, electric-railway, highway, and combined bridges by means of the diagrams, tables, and formulæ of "Bridge Engineering" for rolled I-beam spans, plate-girder spans (both deck and half-through), riveted-truss simple-spans, pin-connected simple-spans, swing spans, cantilevers, suspension bridges, and reinforced-concrete structures; and the results when tested have been found to be exceedingly accurate. The estimating of costs of structures in this manner is a very easy task compared with the job of making similar computations

before these records were compiled; consequently it is to be hoped that bridge engineers will soon learn to utilize properly the vast amount of information collected by the author in his many years of practice as a bridge specialist and presented to the engineering profession in his *magnum opus.*

The following examples and their solutions will illustrate how the diagrams are to be used and how quickly they give results:

Example No. 1

What is the weight of structural steel in a single-track, steam-railway, Class 60, riveted-truss bridge built of carbon steel and consisting of one 420′ through span, two 265′, two 215′, and two 165′ deck spans for a proposed crossing of the Missouri River?

Solution

From Fig. 55*i* on p. 1228, and Fig. 55*l* on p. 1231 of B.E., we have the following:

1	420′ Thro. span	@	5,450 lbs.	= 2,289,000 lbs.
2	265′ Deck spans	@	2,675 lbs.	= 1,418,000 lbs.
2	215′ " "	@	2,060 lbs.	= 886,000 lbs.
2	165′ " "	@	1,620 lbs.	= 535,000 lbs.

Total metal in superstructure = 5,128,000 lbs.

Example No. 2

A double-track, steam-railway, Class 55 bridge, having a total length of 820′ from end to end of superstructure, is to be built so close to high water that half-through plate-girders will be required. Assuming that there will be 10, 11, 12, or 13 spans of equal length, what will be the total weight of metal for each case?

Solution

Ignoring the small spaces over piers between ends of girders, the various span-lengths will be 82, 74.5, 68.3, and 63.1 feet. Referring to Fig. 55*r*, on p. 1237 of B.E., we find the following weights of metal per lineal foot of structure; 3,860, 3,700, 3,550, and 3,420. The total weights of metal will, therefore be,

3,860 lbs. × 820 = 3,165,000 lbs.
3,690 lbs. × 820 = 3,026,000 lbs.
3,550 lbs. × 820 = 2,911,000 lbs.
3,460 lbs. × 820 = 2,837,000 lbs.

Example No. 3

What is the weight of metal in a double-track, steam-railway, Class 65, pin-connected, Pratt-truss bridge, built of carbon steel and consisting of five 330′ through spans?

Solution

From Fig. 55*bb*, on p. 1246 of B.E., we find the weight of superstructure metal to be 7,350 lbs. per lineal foot of span; hence the total weight of metal will be,

$$5 \times 330 \times 7{,}350 = 12{,}127{,}000 \text{ lbs.}$$

Example No. 4

What is the total weight of metal in a double-track, steam-railway, center-bearing swing-span, 350′ long between centres of end supports, having riveted trusses, and carrying a Class 50 live load?

Solution

From Fig. 55*bb*, on p. 1246 of B.E., we find the weight of structural steel per lineal foot of a 350′ fixed span to be 6,450 lbs.; and in Fig. 55*ee* on p. 1249 thereof is given 83 as the percentage to apply for a swing span of that length and the type stated; hence the total weight of metal will be,

$$6{,}450 \times 0.83 \times 350 = 1{,}874{,}000 \text{ lbs.}$$

Example No. 5

A 540′, riveted, Petit-truss, highway span with paved roadway on reinforced concrete base and with reinforced-granitoid sidewalks, for which the total live load per lineal foot of span for trusses is 3,200 lbs., has a dead load, exclusive of weight of trusses, equal to 7,200 lbs. per lineal foot, the total clear width of deck from out to out being 60′. What is the weight of carbon steel in the two trusses required to carry this loading?

Solution

From Fig. 7*e*, on p. 131 of B.E., we find the coefficient for impact with $n = \frac{60}{20} = 3$ to be 0.05; hence each truss load per lineal foot, exclusive of its own weight, will be as follows:

Live load $= \frac{1}{2} \times 3{,}200$ lbs.	$= 1{,}600$ lbs.
Impact $= 0.05 \times 1{,}600$ lbs. . . .	$= 80$ lbs.
Flooring and metal in floor and lateral systems $= \frac{1}{2} \times 7{,}200$ lbs.	$= 3{,}600$ lbs.
Summation.	$= 5{,}280$ lbs.
Assume weight per foot of one truss to be. .	$= 3{,}000$ lbs.
Tentative total load per lineal foot per truss. .	$= 8{,}280$ lbs.

Referring to Fig. 55*hh*, on p. 1252 of B.E., for a 540′ span the 8,000 lbs. curve gives 3,100 lbs. and the 10,000 lbs. curve gives 3,700 lbs.; hence, by

interpolation, 8,280 lbs. would give 3,184 lbs., showing that the assumed truss weight of 3,000 lbs. per foot was too light. Let us try 3,260 lbs. making the total load 8,540 lbs. Interpolating as before gives 3,262 lbs. As this agrees with the assumed 3,260 lbs., it is correct; and, therefore, the total weight of steel in the two trusses is

$$2 \times 3{,}262 \times 540 = 3{,}523{,}000 \text{ lbs.}$$

Example No. 6

What is the weight of metal in the four shoes of a 380′ span, double-track, steam-railway, riveted-truss bridge for Class 60 live load?

Solution

From Fig. 6*e*, on p. 106 of B.E., we find the equivalent uniform live load to be,

2×6,830 lbs.	=13,660 lbs.
From Fig. 7*c*, on p. 129 thereof, we find the impact to be 18% or	2,460 lbs.
From Fig. 55*y*, on p. 1243 of same, we find the weight of metal per lineal foot of span to be	8,000 lbs.
Flooring for two tracks, say	1,000 lbs.
Total load per lineal foot of span.	=25,120 lbs.

Weight of span = 25,120×380. . . . . =9,546,000 lbs.
Load on one shoe = $\frac{1}{4}$×9,546,000 lbs. . . . . =2,386,000 lbs.

From Fig. 55*mm*, on p. 1257 of B.E., we find the average weight of one shoe to be 18,400 lbs., hence the weight of the four shoes is,

$$4 \times 18{,}400 = 73{,}600 \text{ lbs.}$$

Example No. 7

What is the weight of metal per lineal foot of structure in a single-track, steam-railway, steel trestle, 170′ high, to carry Class 55 live load?

Solution

From Fig. 55*rr*, on p. 1262 of B.E., we find the required weight to be 3,140 lbs.

Example No. 8

What is the average weight of carbon steel per lineal foot of structure in a double-track, steam-railway, riveted-truss, Type A, cantilever bridge to carry Class 70 live loading, the length of main span, measured from center to center of piers, being 1140 feet?

Solution

From Fig. 55*ccc*, on p. 1273 of B.E., we find the required weight to be 20,500 lbs.

Example No. 9

What is the yardage in a concrete pier with rounded ends and a half-inch batter, having no coping, the extreme top dimensions being 8′ and 28′ and the height 52′?

Solution

From Fig. 56*b*, on p. 1302 of B.E., we find the yardage of the two half-truncated cones to be 160; and from Fig. 56*d*, on p. 1304 thereof, we find the yardage of a strip one foot wide to be 19.5; hence the total volume will be

$$160+(28-8)\times 19.5=550 \text{ cu. yds.}$$

Example No. 10

What is the yardage in a column-pedestal 4.5′ square on top, 14′ high, and having a batter of two inches to one foot?

Solution

From Fig. 56*k*, on p. 1311 of B.E., we find the volume to be 25.2 cu. yds

Example No. 11

What is the yardage in a wing abutment for a single-track-railway embankment having side slopes of one and a half to one, the vertical distance from foundation to base of rail being 28′, the height of parapet 7′, that of base 2′, that of coping 1′, and that of wing walls at ends 14′?

Solution

The height from bottom of coping to top of base will be about 17′, the greatest height of wing wall above top of base 25′, and the least height thereof above same 12′.

From Figs. 56*o* and 56*p*, on pp. 1315 and 1316 of B.E., we find the following

Volume of Parapet, Coping, Shaft, and Walls to end of Parapet.	185 cu. yds.
Volume of Portions of Wing Walls extending beyond Parapet, and above elevation of top of Base = 190 − 35.	= 155 cu. yds.
Volume of base to end of Parapet = 2 × 17.	= 34 cu. yds.
Volume of base beyond Parapet = 2(14.2 − 7.2).	= 14 cu. yds.
Total volume of abutment.	= 388 cu. yds.

These examples might be continued so as to include the finding of quantities in retaining walls, both plain and reinforced, reinforced-concrete trestles, and reinforced-concrete arch bridges, but space will not permit; hence the reader is referred for examples of the estimates for such structures to pages 1317-1347 of "Bridge Engineering."

Again, by employing the various formulæ given in Chapter XXVII of that treatise, the cost of any highway or electric railway suspension bridge can be quickly estimated. As stated elsewhere herein, the author in a single working day made thereby a close estimate of cost for a 2,500′-span, highway bridge, including substructure, superstructure, approaches, and accessory works, for a proposed crossing of the Detroit River at the City of Detroit.

The eleven simple examples given should suffice to illustrate the facility with which one who is familiar with "Bridge Engineering" can figure the quantities of materials for all ordinary bridges. Those for other structures can be obtained in a similar manner by the expenditure of somewhat more time, but still very readily.

The following are the four student problems referred to near the beginning of this chapter; and the author again suggests that they be solved by any young engineer who desires to perfect himself in the art of making quick computations for costs of bridges:

Problem A

The following data for a river crossing are supposed to be furnished by a survey for a double-track, steam railway in the northwest corner of the state of Arkansas:

Width of watershed at crossing................	36 miles.
Width of same at a distance of forty (40) miles up-stream	44 miles.
Ditto eighty (80) miles up-stream..	28 miles.
Ditto one hundred and ten (110) miles up-stream....	16 miles.
Intermediate widths are to be directly interpolated.	
Extreme length of watershed above crossing . .	125 miles.
Width of river at a fairly low stage of water when the survey was made	520 feet.
Maximum depth of water at the same time at a point about one hundred and ten (110) feet from the water's edge on the left bank............. .	5 feet.
Average depth of water	3.3 feet.
Greatest observed surface velocity at crossing when survey was made, being the average of four observations	2.55 ft. per sec.
Side slope on left bank where the rock is exposed	three (3) vertical to one (1) horizontal.
Height of top of left bank above water at time of survey.	15 feet.

Side slope on the right bank of the stream, from water's edge to break of bank (a distance of thirty-five (35) feet)......... ....one (1) vertical in five (5) horizontal.
Height of top of right bank above surface of water when survey was made............................ 7 feet.
Width of level portion of top of right bank........ 80 feet.
Falling slope back of right bank for a distance of eight hundred (800) feet averages four-tenths (0.4) of one per cent.
Then comes a dry, level slough, three hundred (300) feet wide; and, finally, there is a rising grade of seven-tenths (0.7) of one per cent for a distance of some eighteen hundred (1,800) feet.
A profile of the crossing is shown in Fig. 17*a*.

Average slope of river for first ten (10) miles up-stream is 1.37 feet per mile; and in each ten-mile stretch beyond it increases regularly by one and one-tenth (1.1) feet per mile.

The stream at times carries considerable drift, but there is no probability of the channel changing.

Borings near the water's edge on the right side, at time of survey, showed five (5) feet of silt, sixteen (16) feet of sand, running from fine at top to coarse at bottom, then gravel very fine at first but increasing in coarseness gradually with the depth, the vertical measurements being made from the elevation of the water.

Material of the low bank and of the flat is sandy loam covered with vegetation that would offer considerable resistance to scour. Across the slough the material is harder.

The crossing is near the middle of a long, easy bend in the stream; and the current at high water impinges against the rocky bank.

Highest water-mark found was about eight feet above the water level at time of survey. No reliable records of floods were obtainable.

Rainfall on watershed averages about forty-five (45) inches per annum.

Grade line on structure, twelve (12) feet above the extreme future high-water.

Clearance line for superstructure, at least four (4) feet above same.

Crossing is entirely on tangent and, as nearly as may be, at right angles to the current.

Superstructure is to be of steel, and substructure of concrete.

Piles may or may not be used for foundations.

The channel pier foundations must go to a depth of twelve (12) feet below greatest probable scour in case piles are employed, or twenty (20) feet below same in case that they are not.

Approaches are to be of earth embankment, but it is permissible to put in some wooden pile trestle across the slough, if investigation should indicate it to be necessary.

There are no restrictions as to span-lengths or locations of piers, because the river is not navigable.

The type of floor is the ordinary one of untreated wooden ties and guards with eighty (80) pound rails.

The live load is Waddell's Class 60.

The following are the costs of the various materials delivered at site:

Cement two dollars ($2.00) per bbl.

Broken stone and gravel, one dollar and fifty cents ($1.50) per cubic yard.

Sand (clean and ready for use), one dollar ($1.00) per cubic yard.

Timber, twenty-two dollars ($22.00) per M. feet B. M.

Piles, twenty (20) cents per lineal foot.

Structural steel, 4.5 cents per pound for truss spans and 4.2 cents per pound for plate-girder spans.

Rails, forty-two dollars ($42.00) per long ton.

The following are the schedule costs of the erection work:

Sinking of pneumatic caissons, five dollars ($5.00) per cubic yard.

Sinking by open-dredging, three dollars and fifty cents ($3.50) per cubic yard.

Driving piles for trestle, twenty (20) cents per lineal foot.

Driving piles for foundations, forty (40) cents per lineal foot.

Erection of metal, including cost of falsework and painting, 1.2 cents per pound for truss spans and 0.8 cent per pound for plate-girder spans.

The problem is to determine the extreme high-water profiles both before and after the structure is completed, to make an economic layout of spans, piers, and abutments for the proposed bridge, and to prepare an estimate of cost of the finished structure, exclusive of the earth embankments.

Problem B

Given the profile for a dry-gulch crossing, as shown in Fig. 17*b*, to determine, for a combined highway-and-electric-railway, reinforced-concrete arch bridge, having a clear roadway of forty-four (44) feet and two sidewalks each eight (8) feet wide in the clear, the quantities of all the materials in the structure and an estimate of its cost excluding the earth fills.

The live loads are to be taken as follows:

For the electric railway, Class 25.
For the wagonways, Class B
For the sidewalks, Class C.

Permissible pressure on foundations, five and a half (5.5) tons per square foot.

Arches are to be of the ribbed type with two (2) lines of ribs.

Average depth of excavation for foundations is to be six (6) feet.

For unit prices of materials in place, the average costs given in Tables

57*a* and 57*d* of "Bridge Engineering" are to be employed, except that the cost for concrete in arch ribs is to be taken as $17.00 per cubic yard.

Problem C

Given the profile for a crossing, as shown in Fig. 17*c*, to determine the economic layout and the total weight of structural steel required for a double-track, steam-railway trestle to carry Class 55 live load.

Problem D

Given the profile for a crossing, as shown in Fig. 17*d*, to determine the economic span-lengths and to prepare a complete estimate of cost for a reinforced-concrete-girder structure to carry a thirty-six (36) foot roadway and two (2) sidewalks each six (6) feet wide in the clear, the roadway being paved with creosoted-wood blocks. An earth fill is to be used at each end, but the toes of the front slopes are not to extend beyond the points marked A and B on the profile. Abutments will not be employed, the end columns being buried completely in the embankments. The live loads are to be Class A for the roadway and Class B for the sidewalks. The permissible pressure on the foundations is to be three (3) tons per square foot. The depths of the footings below ground are to average six (6) feet.

The slopes for the fills are to be one and a half (1.5) horizontal to one vertical.

For unit prices of materials in place, the average costs given in Tables 57*a* and 57*d* of "Bridge Engineering" are to be employed.

These four problems were specially chosen for the purpose of making the competitors proficient in the quick computation of approximate quantities of materials and costs of structures, and to train their judgment in the important matter of the determination of best possible layouts for bridges.

CHAPTER XVIII

ECONOMIC SPAN-LENGTHS FOR SIMPLE-TRUSS BRIDGES ON VARIOUS TYPES OF FOUNDATION

UNDER the caption of this chapter there was delivered by the author on September 15, 1919, before the Western Society of Engineers a paper based upon some two hundred *bona fide* special estimates of cost and illustrated by thirty-six diagrams. These illustrations are interesting in that they show graphically how the economics for various types of structure vary with the depth of the foundation; but it has not been thought necessary to reproduce here more than a single set (four) of them and one additional diagram (Fig. 18*h*) in which all the results have been combined in a general way by ignoring certain small, abnormal variations caused by slight irregularities due to the employment of special instead of general data in making the calculations.

The paper reads as follows:

Up to the present time the general knowledge possessed by the engineering profession concerning economic span lengths for bridges has been rather crude and unsatisfactory. Until three decades ago the only data available on this subject were covered by the broad statement that the greatest economy in a bridge layout exists when the cost of a span is equal to the cost of a pier. In his pamphlet on "General Specifications for Highway Bridges of Iron and Steel," issued in 1888, the author pointed out the fact that the then popular impression concerning this question was incorrect, because the cost of the floor is constant, and hence the adjustment is one between cost of substructure and cost of metal in trusses and laterals: Three years later he gave, in a paper published by "Indian Engineering," a mathematical demonstration of the theory of the economics of bridge layouts, showing that the greatest economy will exist when the cost of a pier is equal to one-half of that of the trusses and laterals of the two spans which it helps to support. This demonstration was based upon the assumptions that the piers rest on hard material at moderate depth and that, in most cases being of minimum size, they would not vary in dimensions or total cost for small changes in the span-lengths.

This principle, though, is not applicable to the case of piers resting on sand or on piles, because the cost per lineal foot for substructure is often nearly constant for all moderate span-lengths, while that for the superstructure augments; and this fact is not at all generally recognized by bridge designers. It has become evident of late to the author by reason of

some important bridge studies which he has been called upon to make in his practice, that there is needed by the profession a systematic investigation to determine in an authentic manner the economic span-lengths for simple-truss bridges to support the different kinds of live loads by piers resting on various types of foundation at all practicable depths, and to conform to changing market-prices for materials in place.

In connection with the series of economic studies on bridge design which the author has been making, especially of late years, and which he hopes to complete before he passes on, this question had to be settled sooner or later, consequently he has just spent three weeks in computing the actual costs of both substructure and superstructure for over two hundred cases of bridge layouts covering the following combinations:

Railway, Highway, and Combined-Railway-and-Highway Bridges on Concrete Pier-Shafts overlying Caissons or Cribs resting on Sand, Bed Rock, or Piles, and reaching to depths below low water of 50, 100, 150, 200, and 250 feet; also for low, medium, and high conditions of the material market.

The fact that all the computations were prepared by the author alone, and without a detailed check on the figuring, need not cause any doubt about the correctness of the results of his work, because all of them were plotted on cross-section diagrams, and, consequently, whenever any error of the least importance was made it was detected at once.

This investigation owes its existence to the fact that recently the author as a member of the Board of Advisory Engineers to the Public Belt Railroad Commission of New Orleans (appointed to study the question of bridging or tunneling the Mississippi River at or near that city), had occasion to make a large number of layouts with cost estimates for railway, highway, and combined-railway-and-highway bridges having sand foundations two hundred and fifty feet below the Gulf level. While the conditions precedent for those computations were used for certain of the layouts of this investigation, the actual results thereof were not incorporated, because all the calculations involved in this paper were special and had to be systematized. However, there were numerous deductions made from the New Orleans Bridge studies, which permitted the adoption of valuable short cuts in figuring.

A large portion of the data employed in making estimates of cost was taken from the various diagrams given in "Bridge Engineering," including live loads, impact, and weights of metal.

The following are the assumptions and conditions precedent adopted for the series of calculations:

Character of Structures

The different classes of bridges covered are Double-Track-Railway, Single-Track-Railway, Standard-Highway, and Combined Double-Track-

Railway-and-Highway, all metal being carbon steel (excepting in one set of estimates where nickel steel was employed), the railway floors being open, the highway floors being paved with creosoted blocks resting on a reinforced-concrete base, the foot-walks being slabs of reinforced granitoid, and the handrails being of steel.

The highway bridges considered are all of the author's adopted standard type, viz., carbon-steel trusses, laterals, and floor-system with a 42-foot paved roadway supported on a reinforced-concrete base, two 8-foot sidewalks of reinforced granitoid carried on cantilever brackets, and two steel handrails, making the deck about sixty feet wide from out to out, exclusive of the space occupied by the trusses in through bridges.

All pier-shafts are of plain concrete with a coping, the batter being 1″ to 1′ for low-level-railway and low-level-combined bridges, $\frac{3}{4}$″ to 1′ for high-level-railway and high-level-combined bridges, and $\frac{1}{2}$″ to 1′ for highway bridges.

All caissons founded on sand are of timber with concrete filling and having steel bases and cutting edges; and they are made as light as is legitimate by omitting to fill a large proportion of the excavating shafts. But when the caissons reach bed rock they are assumed to be filled solid. The depth of water in each case is taken as one-third of the vertical distance between extreme low water and caisson footing.

In the pile piers the piles are seventy-five feet long and project sixty feet below the bases, which are assumed to be twenty feet high, the piles being spaced three feet from center to center.

The character of the materials passed through during the sinking is assumed to be the ordinary mixture of silt, quicksand, soft gumbo, and other river deposits, overlying either coarse sand suitable for foundations, or bed rock.

METHODS OF PIER SINKING

The methods assumed for sinking the caissons are those of open dredging and the pneumatic process, the former being employed when the bases are to rest on sand and the latter when they are to reach bed-rock. In the case of pile piers, the open box is first to be sunk by dredging to the required depth, then the piles are to be driven inside of it, and finally the remaining space is to be filled with concrete.

SPECIFICATIONS FOR DESIGNING

The specifications for the designing of superstructure are those given in Chapter LXXVIII of "Bridge Engineering," and those for the designing of substructure are to be found in Chapters XXXIX to XLIII, inclusive, of that treatise.

LOADS

The live loads for superstructure for the several kinds of bridges are given on Fig. 18*a*, and those for substructure on Fig. 18*b*. The former include impact allowances, while the latter do not. Fig. 18*c* records the

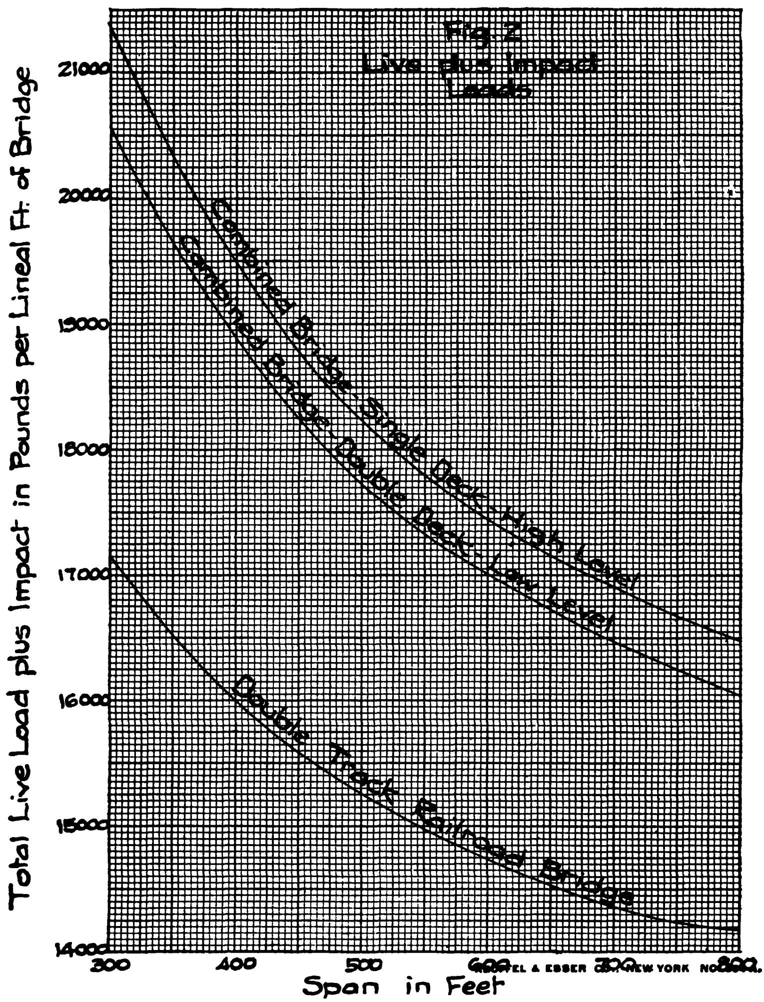


FIG. 18*a*. Live-Plus-Impact Loads.

weights of metal per lineal foot of span in the superstructures of the various kinds of bridges considered. The live loads for highway and combined-railway-and-highway bridges include the proper allowances for electric-railway cars or trains.

The weights per lineal foot for the flooring are as given in Table 18*a*.

Permissible Pressures on Soil and Piles

For sand foundations the method of determining the permissible pressure beneath the base of the caisson is that evolved by the author in making his before mentioned computations for the New Orleans Bridge

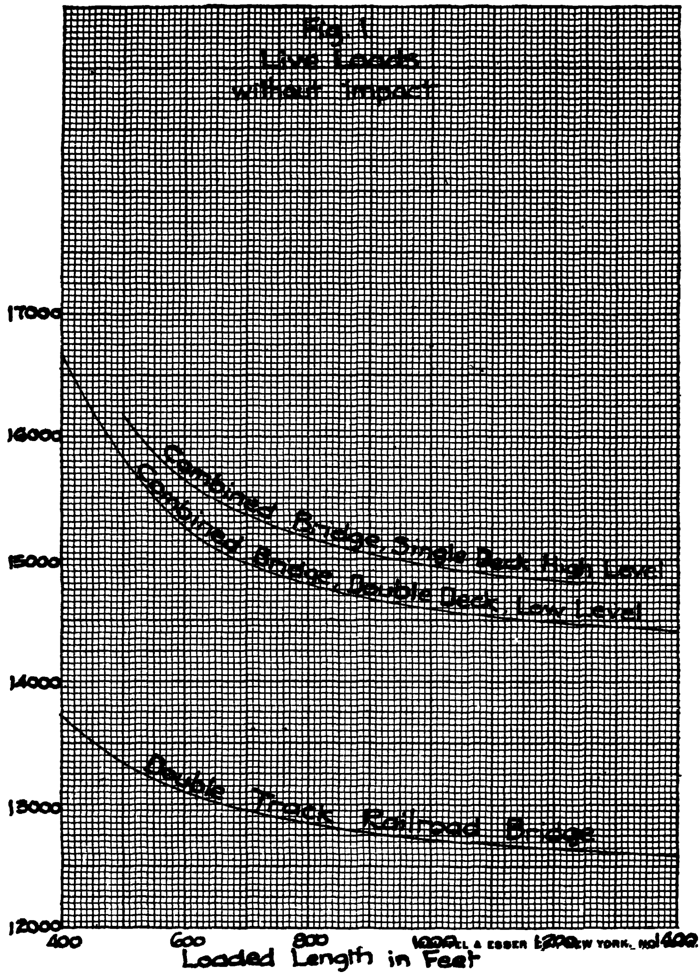


Fig. 18*b*. Live Loads Without Impact.

study. It consists of allowing four tons per square foot plus the intensity of pressure on the adjacent soil at the elevation of the base, due to the *net* weight of the overlying solid material, after having deducted from the net weight of the caisson and its superimposed load for side friction at the rate of 400 pounds per square foot of lateral surface in contact with solid material. The net weight of the water-soaked timber in the caisson is taken as zero

and that of the concrete at eighty pounds per cubic foot. The partially-filled caissons when complete weigh about fifty-six pounds *net* per cubic foot.

As a matter of precaution, the caissons have to be figured for side-frictional resistance of 600 pounds per square foot during sinking, or some-

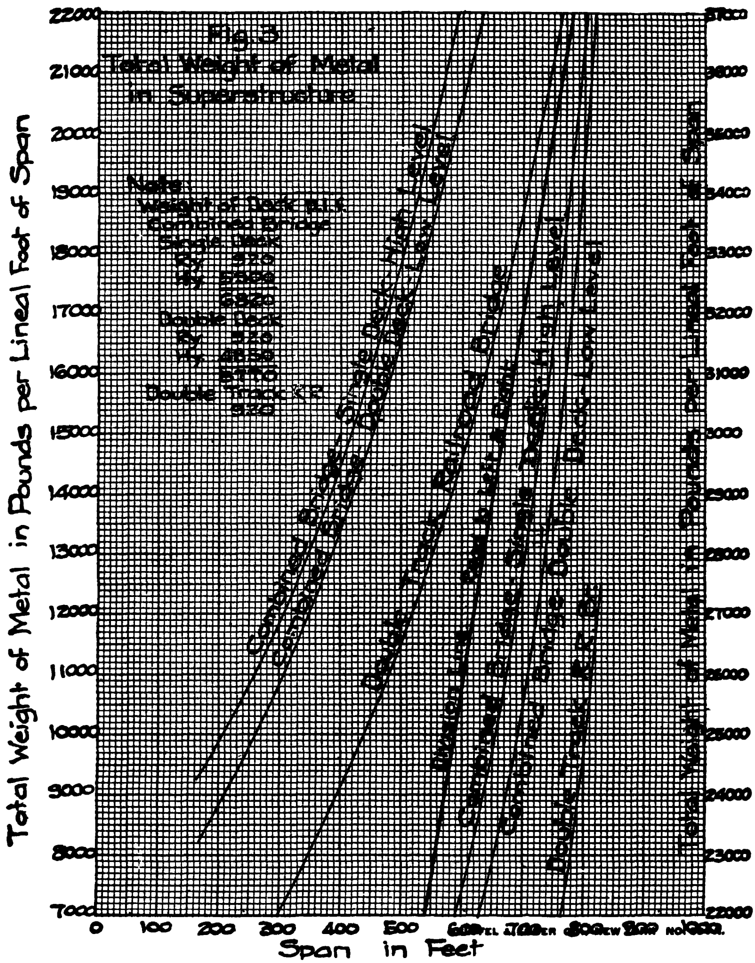


FIG. 18*c*. Total Weights of Metal in Superstructures.

times (in extreme cases) 500 pounds per square foot. Of course, it is practicable to load temporarily the caisson as it reaches the neighborhood of its final position; but such an expedient is sometimes costly and troublesome, hence it is better to design it large enough to avoid the probability of holdup.

Some engineers have objected to relying upon side friction in supporting the load, but their contention is wrong, because it certainly does exist, and it has to be overcome before any settlement of the finished pier can

occur. In the case of long piles driven into soft material, it is almost entirely the side friction which gives them supporting power. Again, someone may question the correctness of loading sand *apparently* as high as nine tons per square foot at a depth of 250 feet below low water level, when the depth of water is eighty feet; but it must be remembered that the *net* weight of 170 feet of earth loads the soil some five tons per square foot, and that before any settlement can occur, the material adjacent to the caisson has to be raised. The reason for this is that the sand at such a great depth is practically incompressible, so that for any settlement to occur it must flow. It cannot flow downward or laterally, because there is no vacant space for it to fill; consequently, if flow it must, it will have to pass upward; and, in order to do so, it must lift a large column of the adjacent solid material. In the author's opinion, it would take an excessively large unit loading on the base of a filled caisson resting on coarse sand at a depth of two hundred and fifty feet to cause the slightest settlement.

TABLE 18*a*

Character of structure	Weight per Lineal Foot for Flooring Exclusive of all Steel but Reinforcing Bars
Low level, combined bridges	5,800 pounds
High level, combined bridges	6,900 pounds
Double-track-railway bridges	900 pounds
Single-track-railway bridges	450 pounds
Standard highway bridges	6,100 pounds

The permissible loading for long piles has been taken at forty tons per pile, this being in accordance with the author's practice for a quarter of a century; and he has never yet found any settlement to occur under such loading.

Unit Prices of Materials in Place

The following table gives the unit prices for materials in place assumed for the purpose of this investigation:

TABLE 18*b*

Materials	Condition of Market		
	Low	Medium	High
Structural steel, per pound	4¢	6¢	8¢
Concrete shafts of 20′ average thickness, per cubic yard	$9 00	$12.00	$15 00
Mass of caissons, including all materials, for a width of 30′ and a height of 150′, sunk by open-dredging, per cubic yard	15.00	20.00	25.00
Mass of cribs, including enclosed pile-heads, per cubic yard	15 00	20 00	25 00
Portion of long piles projecting below base of crib, per lineal foot	75	1.00	1 25

For the "Medium Condition of Market," the price per cubic yard of the shafts is to be modified by the addition or subtraction of fifteen cents for each foot of variation from the assumed average of twenty, the greater the thickness the smaller the unit price. For instance, if a shaft were 12 feet wide under coping and 18 feet wide at the bottom, the average width would be 15 feet and the unit price for medium market \$12.75.

For the same market condition the unit price for mass of caissons is to be modified by the addition or subtraction of ten cents for each foot of variation from the assumed average of thirty, the wider the caisson the smaller the price per cubic yard. Again, for the said market condition, the unit price for mass of caissons is to be modified by the addition or subtraction of two cents for each foot of variation from the assumed average height of one hundred and fifty feet, the deeper the caisson the smaller the unit price. For instance, with medium condition of market, the unit price for a caisson twenty-six feet wide and two hundred and forty feet high would be

$$20.00+4\times0.10-90\times0.02=\$18.60.$$

For the other two assumed conditions of the market, these figures of modification would have to be multiplied by the ratios indicated in the table, viz., 0.75 and 1.25.

Without these modifications of unit prices for substructure, the investigation would be not only illogical, but incorrect. The variation in cost of shafts per cubic yard is due primarily to the lower unit cost of forms for thick piers, but also somewhat to the economy effected by manufacturing and handling larger masses of concrete. The latter reason applies also to the two variations in the cost of mass of caissons; but the main cause thereof is that the total cost of cutting edge, shelter against current, and flotation to final location are the same for a shallow base as for a deep one.

The prices per cubic yard for caissons sunk by the pneumatic process, under medium-market conditions, have been made two dollars greater than those for caissons sunk by open-dredging. This is in conformity with the author's bridge experience of nearly four decades. It is due primarily to the more rapid sinking by open dredging and the greater cost of the pneumatic outfit, but also to the fact that the pneumatic caissons are generally filled solid, while the open-dredging caissons often have their excavating wells only partially filled.

The price used for nickel steel superstructure in place for medium market conditions has been taken as eight and a half cents per pound; for the reason that the last *ante-bellum* figures quoted to the author made the price of nickel-steel two and a half cents per pound higher than that of carbon-steel. The weights of metal in nickel-steel superstructures were computed by means of ratios determined from diagrams given in the author's paper "Nickel Steel for Bridges."*

* See Trans. Am. Soc. Civ. Engrs. for 1909.

Method of Determining the Economic Span-Lengths

In determining the economic span-lengths, computations were made for the volumes of concrete in shafts, volumes of caissons, volumes of cribs, total lengths of piles below crib bases, and weights of metal in spans, but no notice was taken of the cost of flooring, as that is a constant for any type of bridge.

It might be well to mention that while the abscissæ of the diagrams give the span lengths measured from center to center of end pins, the costs of structure per lineal foot were computed by using the distance from center to center of piers.

In making each of these cost estimates there was assumed a structure of indefinitely great length and unvarying profile, so that the sum of the cost of the steel work in a span and the cost of a complete pier divided by the horizontal distance between adjacent-pier centers gives the comparing cost per lineal foot of structure, although, as before indicated, not the *complete* cost thereof.

The results of all calculations made were plotted on cross-section diagrams, but only a few thereof have been reproduced herein. However, the important deductions from all the estimates have been tabulated. The plotting was done with the utmost care, and due consideration was given to a proper determination of the economic span-length. As previously indicated, a number of arithmetical errors were located and corrected by reason of irregularities in the curves, thus making the latter truly reliable. In almost all cases, at least four points were plotted from computations, in order to locate the curves of cost for substructure and for the steelwork of superstructure; and a combination of these was used for locating a few intermediate points on the curve which gives the combined cost of substructure and steelwork. In a few instances, though, three points for the lower curves were found to be sufficient for a correct plotting of the upper curve.

Recording Diagrams and Table

On Figs. 18*d* to 18*g*, inclusive, are graphically recorded specimen diagrams of the results of the special calculations. Each diagram contains three curves, one for substructure, one for steelwork in superstructure, and the other for a combination of these two. The computed cost points therefor are marked on the three curves, respectively, by circles, squares, and diamonds. The abscissæ of these diagrams give the span-lengths in feet, measuring from center to center of bearings; and the ordinates record the cost per lineal foot, measuring from center to center of piers. On each diagram is clearly indicated the span-length for greatest economy; and it is to be noticed by the flatness of the upper curves that a variation of twenty-five feet or more, either above or below the economic length, will make very

little difference in the cost per foot of structure. Each diagram is provided with a title which indicates clearly the type of structure and depth of foundation to which it refers. Unless otherwise shown thereon, these

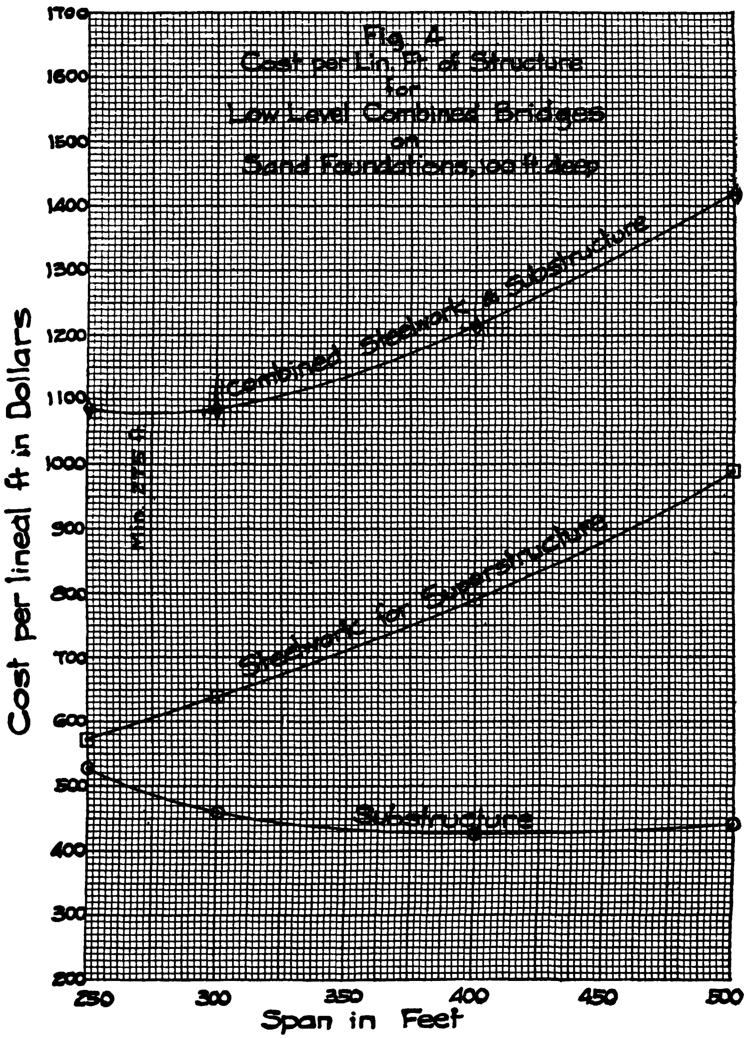


FIG. 18*d*. Costs per Lineal Foot of Structure for Low-Level, Combined Bridges on Sand Foundations 100 Feet Deep.

diagrams relate to normal or medium conditions of the material market.

In the following table is given a résumé of the results of most of the cost computations that were prepared:

TABLE 18c

RÉSUMÉ OF RESULTS OF COMPUTATIONS

Character of Structure	Character of Foundations	Depth of Caisson Footings	Economic Span Lengths	Remarks
Low-Level Combined	Sand	100′	275′	
Low-Level Combined	Sand	150′	300′	Shaft Batter
Low-Level Combined	Sand	200′	325′	1″ to 1′
Low-Level Combined	Sand	250′	350′	
Low-Level D. T. R. R.	Sand	100′	275′	
Low-Level D. T. R. R	Sand	150′	310′	Shaft Batter
Low-Level D. T. R. R.	Sand	200′	360′	1″ to 1′
Low-Level D. T. R. R	Sand	250′	430′	
High-Level Combined.	Sand	100′	275′	
High-Level Combined.	Sand	150′	300′	Shaft Batter
High-Level Combined.	Sand	200′	325′	$\frac{3}{4}$″ to 1′
High-Level Combined	Sand	250′	350′	
Low-Level Combined	Rock	50′	250′	Pneumatic
Low-Level Combined.	Rock	100′	300′	Caissons
Low-Level D. T. R. R.	Rock	50′	275′	Pneumatic
Low-Level D. T. R. R	Rock	100′	325′	Caissons
High-Level Combined.	Rock	50′	300′	Pneumatic
High-Level Combined.	Rock	100′	350′	Caissons
Low-Level S. T. R. R.	Rock	50′	250′	Pneumatic
Low-Level S. T. R. R.	Rock	100′	300′	Caissons
High-Level Combined.	Piles	20′	175′	Pile Piers
Low-Level Highway.	Sand	100′	300′	
Low-Level Highway.	Sand	150′	350′	Shaft Batter
Low-Level Highway.	Sand	200′	400′	$\frac{1}{2}$″ to 1′
Low-Level Highway.	Sand	250′	450′	
High-Level Highway.	Sand	100′	325′	
High-Level Highway.	Sand	150′	350′	Shaft Batter
High-Level Highway.	Sand	200′	375′	$\frac{1}{2}$″ to 1′
High-Level Highway.	Sand	250′	400′	
Low-Level D. T. R. R	Sand	100′	350′	
Low-Level D. T. R. R	Sand	150′	385′	Nickel-Steel
Low-Level D. T. R. R	Sand	200′	425′	Super-
Low-Level D. T. R. R	Sand	250′	470′	Structure
Low-Level D. T. R. R.	Sand	100′	290′	
Low-Level D. T. R. R	Sand	150′	330′	Low-Market
Low-Level D. T. R. R	Sand	200′	375′	Unit-Prices
Low-Level D. T. R. R.	Sand	250′	425′	
Low-Level D. T. R. R.	Sand	100′	275′	
Low-Level D. T. R. R	Sand	150′	325′	High-Market
Low-Level D. T. R. R.	Sand	200′	375′	Unit-Prices
Low-Level D. T. R. R	Sand	250′	425′	

These results are recorded graphically in Fig. 18h.

From a study of the preceding table there can be drawn the following deductions:

A. For all types of bridges the economic span-length increases with the depth of foundation, though not necessarily in the same proportion.

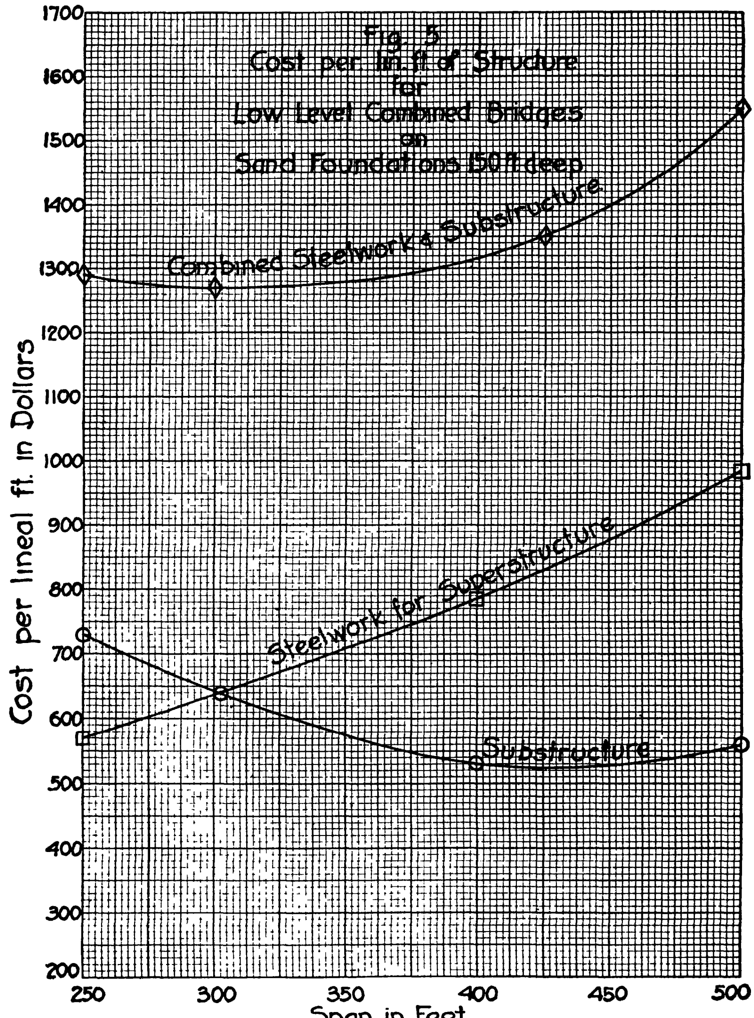


FIG. 18e. Costs per Lineal Foot of Structure for Low-Level, Combined Bridges on Sand Foundations 150 Feet Deep.

B. The lighter the superstructure and the live load it carries, the greater generally is the economic span-length, and the greater the variation of the latter with the depth of foundation.

C. For sand foundations there is not much difference in the economic span-lengths for low-level and high-level bridges of the same type.

D. Structures with piers founded on bed rock generally have economic

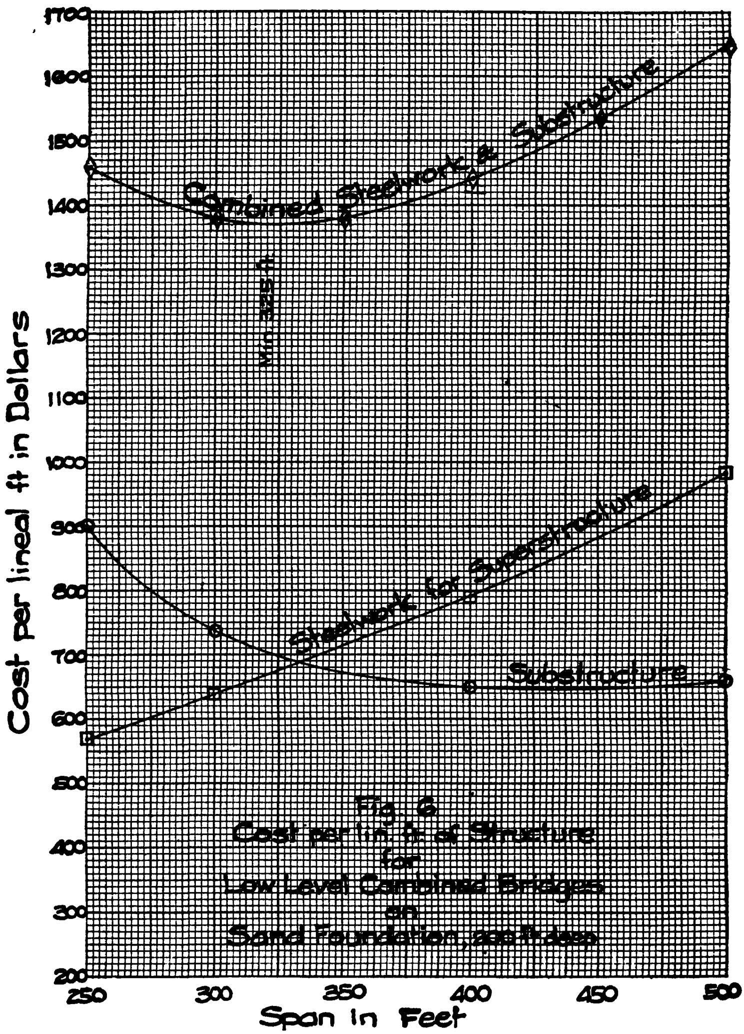


FIG. 18*f*. Costs per Lineal Foot of Structure for Low-Level, Combined Bridges on Sand Foundations 200 Feet Deep.

span-lengths somewhat greater than those of the corresponding structures founded upon sand at the same depth.

E. Single-track railroad-bridges have economic span-lengths a little less than those of the corresponding double-track structures.

F. Pile piers for high-level bridges involve, for economic considerations, rather short spans; and for low-level structures they usually neces-

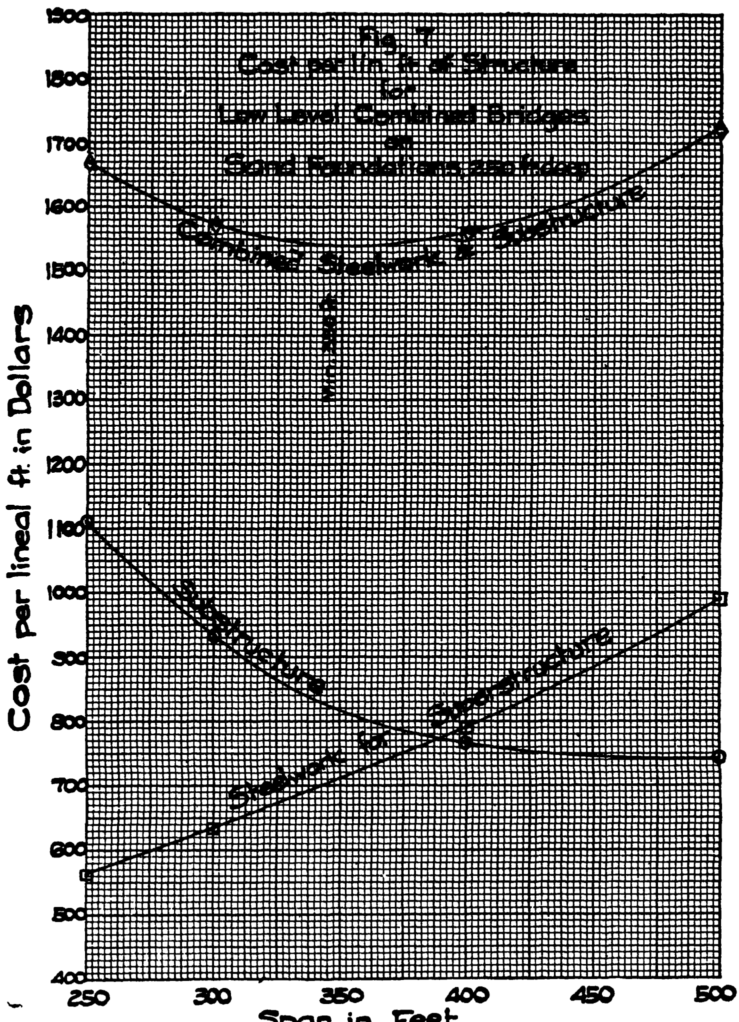


FIG. 18*g*. Costs per Lineal Foot of Structure for Low-Level, Combined Bridges on Sand Foundations 250 Feet Deep.

sitate such short ones as to require the adoption of plate-girder superstructures.

G. In highway bridges having very deep foundations on sand, increasing the batter of the shaft augments the economic span-length.

H. Using nickel steel instead of carbon steel in the superstructure increases materially the economic span-length.

I. The assumed variations in unit prices with changing market conditions make very little difference in the economic span-lengths. There would have been no difference at all had the prices of all the materials used been assumed to vary in the same proportion; but the superstructure steel,

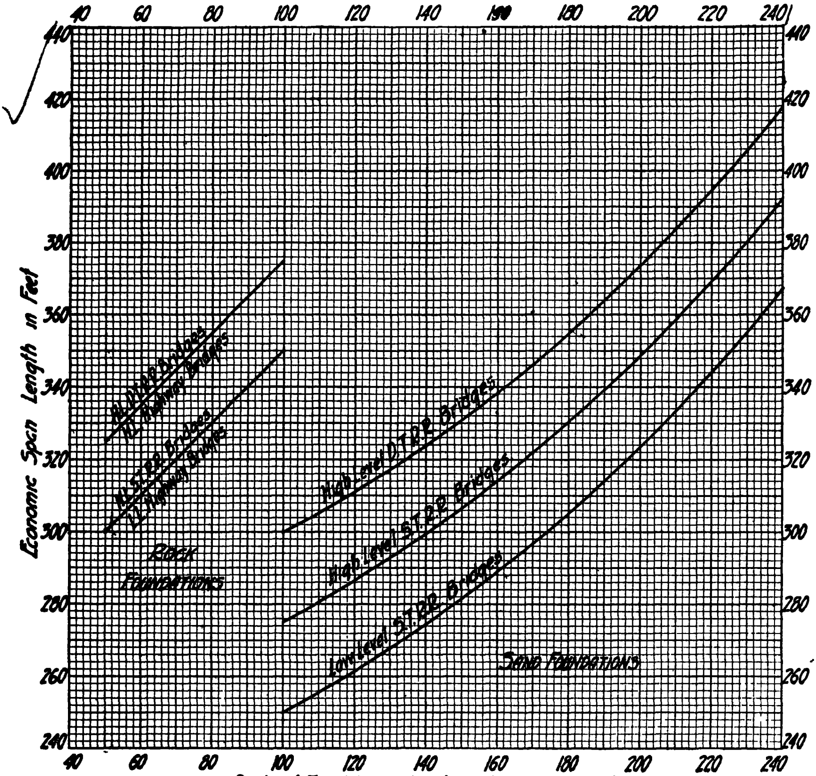


FIG. 18*h*. Economic Span-Lengths for Simple-Truss Bridges on Various Types of Foundation.

erected, ordinarily changes in value somewhat more rapidly than does the substructure of the bridge.

J. There are not many irregularities to be found in comparing the diagrams or the tabulated results of the calculations; and what few exist are small. They are generally due to the adoption of a minimum weight limit for sinking to great depths instead of figuring upon employing temporary loading.

Certain of the cost curves in the diagrams, in combination with other diagrams giving weights of steel distributed between trusses, laterals, and

floor systems, will provide a check on the correctness of the old method of determining economic span-lengths. Let us take the case of a low-level, double-track-railway bridge founded on rock, find the cost per lineal foot of the trusses and laterals in the span of economic length, and check it against the cost per lineal foot for the substructure thereof. For a 50-foot depth of bed rock the economic span-length is 275 feet; and for that span (see "Bridge Engineering," pages 1239 and 1240) the weight of metal per lineal foot for trusses and laterals with Class 60 live load is 4,600 pounds, which at six cents per pound would be worth $276, while the cost per foot for the substructure given by the diagram is $270. This is not a bad check.

For a depth of 100 feet, the economic span-length is 325 feet, for which the weight of trusses and laterals is 5,860 pounds, which at six cents per pound would be worth $352. The diagram makes the cost per foot for the substructure $420—quite a discrepancy.

For low-level, single-track-railroad bridges with a foundation depth of 50 feet, the economic span-length given by diagram is 250 feet, for which the weight of trusses and laterals is 2,480 pounds, which at six cents per pound would be worth $149, while the said diagram gives the cost per foot for substructure at $175—not a close check.

For a depth of 100 feet, the economic span-length is 300 feet, for which the weight of trusses and laterals is 3,050 pounds, which at six cents per pound would be worth $183. The diagram makes the cost per foot for substructure $275—another large variation.

It is evident from the preceding comparisons of cost that the former rule for determining the economic span-length is not reliable, especially for foundations at great depths; hence its use should be discontinued.

There is a little economic, or more strictly speaking uneconomic, investigation concerning simple-truss spans the results of which are worth knowing and may sometimes prove valuable, especially in answering questions propounded by laymen, viz., "what are the relative weights of metal in equal-truss, three-span bridges and structures of the same kind, same total length, and same loading, but having the central span lengthened and the other two equally shortened?" The answer to this question is that the weight-ratios for the unequal-span layouts, as compared with those for equal spans, are greater for long structures than for short ones, and increase with the ratio of middle-span length to average-span length. The values of such ratios for three-span structures, varying in average span-length by one hundred feet from 200 feet to 500 feet, are given in Fig. 18*i*.

The curves for cost ratios are almost coincident with those for weight-ratios, because the pound prices erected for the metal in the various layouts are nearly alike. On the one hand, those for the equal-span layouts should be less because of a saving in cost of making working drawings and templets; but, on the other hand, the erection costs per pound are a trifle less for the layouts of unequal-span length because of their greater total weights of metal. C. W. Bryan, Esq., Chief Engineer of the American

Bridge Company, has very kindly investigated this question for the author; and in a letter dated January 29th, 1920, he reports as follows concerning a double-track, steam-railway bridge, 1,200 feet long, for which the metal erected is assumed to cost 7.0¢ per lb.:

After careful study I feel that the three equal spans and the two spans of 350 feet with one of 500 feet should take the same pound price. For the two spans of 300 feet

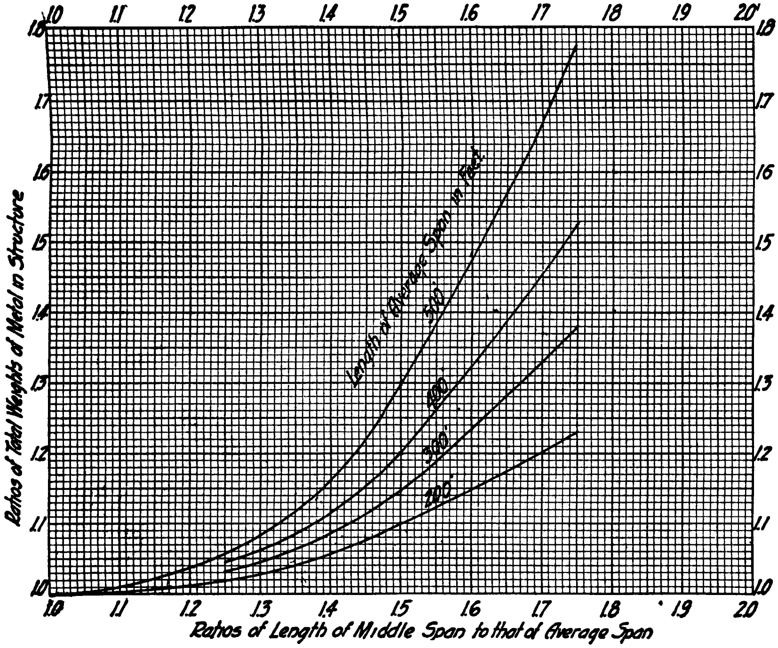


FIG. 18*i*. Diagram of Weight-Ratios showing Effect of Lengthening the Central Span of any Simple-Truss, Three-Span Layout, Keeping the Total Length Unchanged.

and one of 600 feet I would reduce the price $1.00 per ton or to 6.95¢ per pound; and for the two 250-foot spans and one 700-foot span, I would reduce the price $2.00 per ton, or to 6.90¢. per pound.

As the extreme variation in pound price is only one-and-a-half per cent, evidently, as before stated, it is not worth while to make a special diagram recording total cost-ratios for uneconomic, three-span, simple-truss layouts; because those for the weight-ratios thereof will suffice.

CHAPTER XIX

ECONOMICS OF SUBSTRUCTURES

In the old days of cut-stone-masonry piers, the method of proportioning the shafts was to make them as small as possible on top, keeping the pedestals of the spans just within the periphery limits of the first sub-coping course, putting on a batter of half an inch to the foot, and carrying this down as low as the governing conditions would permit, thus ignoring entirely the effect of thrust from trains or wind. For the small structures of those times, with their short spans, this arrangement generally answered the purpose well enough, because the maximum theoretical thrusts assumed in modern bridge practice seldom, if ever, came upon the structures; but occasionally there would arise a case for which this rule-of-thumb method of pier-shaft proportioning would not suffice.

The author recalls an experience of the late eighties when at Albuquerque, New Mexico, he was making the preliminary calculations for the Red Rock cantilever bridge over the Colorado River. The Chief Engineer of the railroad company had undertaken the designing of the piers, and after learning the area required for the main pedestals, he proportioned the coping, then laid out the rest of the shaft with a batter of one-half inch to the foot, and submitted the design to the author for comment. A glance brought the instant conclusion that something was wrong, and upon this being stated the job of pier-designing was turned over to him, whereupon he proceeded to figure the overturning effect on the pier by combined traction and wind loads, with the result that a batter of one and a quarter inches to the foot was found necessary; and this batter gave the layout a decidedly-pleasing appearance.

The day of cut-stone-masonry piers is past—or, at any rate, ought to be; for compared with concrete piers they are always uneconomical. Sometimes, as a defence against the grinding effect of ice, or the disintegrating effects of sea-water between high-water and low-water levels, it is necessary to protect the concrete thus exposed with a facing of granite or other hard rock; and occasionally someone desires to adhere to cut-stone work for the sake of retaining the old-fashioned appearance which it gives to structures; but no engineer who is a student of true economy in design and construction will continue to use coursed masonry in his bridge piers.

There is an economic problem in concrete-pier designing which comes up occasionally—whether it is better to reinforce for bending due to traction and wind loads or to omit the rods and use more concrete. There is no

means of settling this except to design the shaft in both ways and compute the costs. Even then it may not be economics but aesthetics which will govern the decision, because the reinforced-concrete piers are liable to lack the massive appearance which is necessary for a pleasing effect. Unless there be some really-material advantage in reinforcing the shafts, it is generally better to build them of plain concrete.

There is an economic expedient in pier designing that is very often perfectly legitimate, especially in small structures and occasionally in very large ones, viz., the use of the "dumb-bell" cross-section, or, in other words, adopting two pedestals with a diaphragm wall between. This wall may either rest on a continuous base or may be entirely unsupported between the pedestal bases, thus acting as both a strut and a beam. The appearance of a structure having piers of this type is not unpleasing, and the effect of massiveness is obtained by the expenditure of very little extra material. In a wide, two-truss bridge, solitary pedestals without a connecting wall may be employed, reliance being placed upon the end floor-beams of the spans to divide the wind load about equally between the two supports. In case that the deck is fairly close to the water, the great width will partially hide the substructure, and the lack of the connecting wall will not be noticed; but in a high-level bridge, especially when carrying railroad trains, pedestal shafts not only produce a flimsy appearance but also fail to resist properly the rack from the live load. The Missouri River bridge at Glasgow was originally built in that manner; and the experience with its piers was so unsatisfactory that the twin pedestals had to be removed and replaced by a solid shaft.

In some cases it is essential that the load on the foundations be reduced to an absolute minimum, and to this end hollow shafts, or pedestals connected by two thin walls, may be employed; and the excavating shafts in the bases need not be filled, excepting only sufficiently at the bottom to transfer properly the upward thrust of the foundation into the solid portions of the base.

Tall steel cylinders filled with concrete and well braced between make an economical substructure for light highway bridges; and this type of construction is proper, provided that the cylinders be carried far enough down into fairly-hard material to hold firmly the lower ends, so as to enable the cylinders to act as beams with fixed ends for resisting the bending effects of wind loads and traction loads. Generally in such cases it will be found necessary to build a substantial mattress around each pier, so as to prevent the scouring out of the material upon which reliance is being placed for fixing the ends.

Temporary piers of timber, such as those built by the author in the middle nineties for the Missouri River bridge between Council Bluffs, Iowa, and East Omaha, Nebraska, are a legitimate economic expedient, provided that due arrangement be made for replacing them later by permanent piers without involving any unnecessary expense or interrupting

traffic. In building a new railroad through timber country it is in the line of true economy to put steel spans temporarily on pile piers in order to avoid the excessive cost of hauling in substructure materials before the track reaches the site. In such cases the temporary piers should be located far enough from the positions of the permanent piers to allow the latter to be constructed without interfering with traffic.

The employment of the cocked hat in pier shafts is generally an architectural extravagance that should be avoided whenever its use is not demanded by the necessity for spreading the base over a wide pile foundation. It certainly relieves the monotony of appearance in a tall shaft, but the principles of economy generally bar it out. Only once in the author's career has he ever been guilty of adopting the cocked hat, viz., in the late eighties when he made the design for what was then termed the Winner Bridge over the Missouri River at Kansas City, the spans of which were proportioned to carry between the trusses a single-track railway with a narrow foot-walk on each side and a single-track roadway outside of each truss, thus making the perpendicular distance between central planes of trusses twenty-five (25) feet. It proved to be a fortunate thing that the cocked hat was employed, for the superstructure of the Winner Bridge was never erected, because of lack of funds; and when the double-deck, double-track, Fratt Bridge was built on the old piers, after cutting down the shafts to an elevation of ten feet above high-water mark, the extra length of pier afforded by the said cocked hat provided just the necessary extra size for permitting the superstructure to be widened sufficiently to carry the double track.

Ice breakers are sometimes used where there is no real need for them, because it takes an enormous amount of thick ice to damage any well-founded pier having a shaft with rounded ends; nor, as a rule, does a concrete pier require special protection against the grinding of ice. If real granitoid of one-two-three composition with ten per cent of hydrated lime added to the cement were substituted for the ordinary concrete between high-water and low-water marks and extending into the mass about twelve inches, the protection thus afforded would almost always be ample and would involve very little additional expense. Moreover, the repairing of an abraded pier-surface by means of granitoid is not a difficult matter. Unless a pier rest on bed rock, the placing of an unbalanced ice-break upon it is going to upset the equality of load distribution over the foundations and thus, possibly, cause trouble. In case of a pier on piles, it would be better to put another ice-break on the down-stream end of the pier for the sake of symmetry, although it would serve no useful purpose as an ice-break *per se*. All violations of the precept of symmetry are to be avoided whenever this is practicable; for, by so doing, trouble also is often avoided.

The economic question of reinforced-concrete *versus* timber for cribs and caissons is beginning to loom up. At the same total first cost, timber is preferable, owing to the ease and rapidity with which it may be put in place;

but its growing scarcity will certainly make the reinforced-concrete shells of such constructions more and more popular among bridge builders as the years go by. In using it, the time required to let the concrete set and harden is liable to cause delay in the sinking; and the removal of the forms is often troublesome.

This last statement leads to the thought that there is coming up soon the economic question of steel *versus* timber for concrete forms. At present the former material is not much employed for this purpose, but its adoption therefor is on the increase, especially when the forms have to be used a number of times. After timber has been utilized for this purpose three or four times it becomes badly broken up and unfit for further service, while the steel forms with care can be used an indefinitely great number of times. A combination of the two materials might be employed to advantage, the steel being interposed between the concrete and the timber girders, thus avoiding injury to the latter and permitting them to be used over and over again.

In respect to the depth below extreme low water to which it is economic to carry the shaft of a pier, there is great difference of opinion amongst engineers. The author generally locates the plane of division between shaft and base at an elevation of two feet below the lowest-recorded water-level, thus providing against exposure to the air of water-soaked timber, even in seasons of abnormal drought. Such treatment for a short time would probably do no harm, but the exposure of the crib to vision is not pleasing. Those who claim economy for locating this division plane far below the water's surface do so on the plea that it requires less material. This is true enough, but the unit price of the portion of the shaft below low-water is far higher than that above the same, and generally somewhat greater than that of the top of the crib itself. For this there are several reasons, viz.:

First. In order to build it, a fairly-water-tight, removable cofferdam has to be constructed, which is certainly more expensive than the simple crib-top.

Second. This cofferdam has to be kept pumped clear of water until after the shaft is built.

Third. The form-work below low-water is expensive and adds materially to the unit cost of the shaft-concrete.

Fourth. Where the shafts are carried down deep, more allowance has to be made for possible error of position; and to do this would involve the enlarging of the area of the base, thus increasing the total cost.

The condition sometimes exists which calls for the least possible obstruction of the waterway, and then it often becomes necessary to carry the shaft down to the bottom of the channel, irrespective of the extra cost of the piers. In such cases it will be found that the unit value of the shaft-concrete will be high, and that, as far as mere cost is concerned, it would have been better to carry the cribs up to near low-water mark.

The determination of the proper clearance to allow between the bottom of the shaft and the inside of the timber or reinforced-concrete shell is an economic problem of importance. If it be made unnecessarily large, the volume of the base will be too great and the construction too costly. On the other hand, if it be made too small and an error in location should occur because of unanticipated trouble in sinking, it would be difficult to shift the shaft the right amount on top of the crib in order to get it into correct position; and this would involve delay, than which there is nothing more expensive in substructure construction. It is evident that one must endeavor to strike a happy mean in designing his cribs and caissons—but what is that mean? The author's practice is to allow as a minimum a foot clear all around the base of the shaft for easy conditions of sinking, and to increase this gradually as the said conditions become more and more unfavorable, up to a limit of about twice that amount. It would certainly be a case of either gross carelessness or extremely hard luck which would prevent the correct location of a pier-shaft when the larger allowance-limit for shifting was provided. With due care in sinking, the error of position of a crib-top should seldom exceed a few inches; consequently, when a bridge engineer, in order to be surely on the safe side, makes an abnormally-great allowance for error of crib position, he does so at the expense of the work, and therefore imperils his reputation as a true economist.

Whether to use the pneumatic process instead of either open-dredging or cofferdam excavation is fundamentally an economic problem based upon the theory of probabilities. Comparing the open-dredging and the pneumatic methods of sinking, while the former generally figures out to be the cheaper, its cost is rather uncertain, because of the possibility of encountering large logs or boulders; and, while the cost of installation of a pneumatic plant adds some two or three dollars to the cost per cubic yard of the bases, one can count almost with certainty upon the total expense involved in the sinking. If bed rock be within reach by the pneumatic process, that method of sinking should always be adopted, unless it be decided not to go that far down for a foundation, in which case the open-dredging process is likely to be the more economic. One should never sink a caisson to bed rock by open-dredging for fear that it will rest on one edge or one corner only and thus provide an unequal bearing. It would be far better to stop short of it a small distance and rest on sand, gravel, or boulders overlying the rock.

Comparing the cofferdam method with that of open-dredging into a clay or other fairly-hard foundation-material, unless the depth below the working stage of water be less than eighteen (18) or twenty (20) feet, the latter usually is preferable, because the former is likely to give trouble and nearly always involves a greater expenditure of money than that allowed in the preliminary estimate.

The economics of steel sheet piling for cofferdams is still an unsettled question among bridge engineers. Some of the old-time substructure con-

tractors contend that cofferdam piers sunk by their use are more expensive than those placed by the pneumatic process; whilst other contractors, equally experienced, declare the contrary, all however agreeing that the difference in cost by the two methods is small. The author is inclined to believe that, if a thorough set of borings has failed to indicate any sunken logs, beds of large boulders, quicksand, or other serious impediment to driving or excavating, the steel sheet-piling will involve a moderate saving in first cost for depths of foundation below ordinary low water as great as forty feet. To effect such a saving, however, requires an experienced and energetic contractor or superintendent; piling of ample size, thickness, and length; sufficient of it for three, four, or even five piers (according to the size of the job), with extra pieces to provide for damage in driving; heavy pile-driving hammers; ample pumping capacity; and a full supply of derricks, engines, and other outfit. Generally, it is the small-fry contractor who prefers the cofferdam method to the pneumatic; and he is the one who is most likely to get into trouble from failure to anticipate and provide against difficulties in driving and excavating. His pseudo-economic disposition leads him into purchasing small, thin, and short piles; for he does not recognize that large ones will withstand battering at both top and bottom much better than small ones, that thin webs are liable to be split and bent by striking large, hard boulders, and that short lengths are almost sure to involve not only flooding the dams but also filling them with sand or silt—possibly several times during the progress of the work.

Steel-pile cofferdams have been successfully used for depths as great as fifty feet below ordinary-low-water elevation; but the conditions were unusually favorable, the material penetrated being mostly soft clay that shut out the water almost completely, thus enabling bed rock to be reached at moderate expense.

Large, strong sheet-piles, in addition to the security against injury in driving which they provide, effect an economy by permitting the waling frames to be placed farther apart, thus lessening the amount of timber to be bought and the expense of both its placement and its removal; besides the metal often has more than merely scrap value at the end of the job, which is seldom the case when small, light sections are employed.

Sunken logs or wrecks give endless trouble when encountered in cofferdam work, as usually they have to be shattered to splinters by dynamite before the piling can penetrate them; and under such conditions the steel sheet-piling is decidedly superior to the wooden Wakefield-piling. The latter is advantageous for shallow excavations and for cases where the bed rock is too hard to penetrate; because the ends of the piles broom, and the battered wood, by absorbing water, expands and seals the bottom of the pit.

The cofferdam method is specially applicable where clay overlies the bed rock; for it will seal the bottom of the box. Where there is no such sealing layer, it is often necessary to place clay, manure, or some other

material on the outside, or else to employ a double-wall cofferdam filled with clay.

In relation to the comparative economics of a caisson, put down by open-dredging to a depth that is absolutely great enough to insure against disaster from scour, and a crib sunk to a reasonable depth and filled with long piles, the latter generally will be found to be much less expensive; but cases will occasionally occur when the reverse is true, hence it is well always to make complete comparative estimates of cost. These require only a few minutes' work for an experienced bridge computer.

Once in a while it occurs that the pneumatic process has to be employed for one or more piers of a long bridge, the others being sunk by open-dredging. The question then arises as to how many to put down by the more expensive process; and it should be solved more with reference to the expediting of the construction than to using pneumatics only where actually required. For instance, if all the piers but one could be handled by open-dredging, it would be found that the unit price for the base of that one would run extravagantly high, because it would have to take care of a large overhead charge for use and transportation both to and fro of the pneumatic machinery and other outfit. In that case one should figure how much it would actually cost to put down another pier by pneumatics when everything necessary is on the ground and ready to move over to the site of the next pier, then compare the result with the cost of that pier sunk by open-dredging. Generally but little difference will be found, and, therefore, it might prove truly economic to keep both outfits occupied.

The sooner any piece of bridge construction is finished, the sooner will the erection contractor be ready to undertake another contract—besides it is often the case that an expeditious handling of the work will avoid a rise of river or the advent of other unfavorable working conditions. It is, therefore, logical to conclude as a general proposition that the harder a contractor drives his work the more money will he net from his operations, even if it appear to the casual observer that he is spending cash rather recklessly for the purpose of finishing the work quickly. Of course, if the contractor is absolutely sure that he will have no work at all to keep his force occupied after a certain job that he is on is completed, it will not pay him to spend any extra money to rush it; but, on the other hand, there is nothing to be gained by dragging it out unnecessarily. It would be better to finish it and trust to luck about getting another contract.

In foundations for trestles there sometimes arises the economic question whether, in order to obtain the requisite bearing area, it would be better to use plain concrete and go rather deep by stepping off the base in the old-fashioned way, or to spread out quickly by adopting reinforcement. The surest way to settle the question is to proportion a pedestal or two by each method and compare results. If for any reason the cost of the excavation should run high, the reinforcement method will have a decided advantage. Again, as the volume of the concrete is much smaller for that type of base

than for the other, the total load on the foundation is less, and, consequently, the area of base required is smaller. Whether the excavation is wet or dry will make considerable difference, the former condition favoring reinforcing and the latter militating for plain concrete. Where the footing cannot be pumped out, it is rarely permissible to use reinforcement.

Sometimes in a long trestle it is doubtful whether to adopt reinforced-concrete piles with quite-shallow bases for the pedestals or to employ the cheaper wooden piles and sink the said bases to extreme-low-water elevation, in order to ensure that the wood shall always be saturated and never exposed to the air. In such a case, one or two pedestals should be designed by each method and their costs determined for comparison.

CHAPTER XX

ECONOMICS OF TRUSSES AND GIRDERS

DURING the last half century several treatises have been written upon the subject of economy in superstructure design, but unfortunately the result is simply a waste of good mental energy; for the writers thereof invariably attack the problem by means of complicated mathematical investigations, not recognizing the fact that the questions they endeavor to solve are altogether too intricate to be undertaken by mathematics. The object of each investigation appears to have been to establish an equation for the economic depth of truss, or that depth which corresponds to the minimum amount of metal required for the said truss; and, to start the investigation, it seems to have been customary to make certain assumptions which are not even approximately correct. For instance, the principal assumption of several treatises in French and English is that the sectional area and the weight of each member of a truss are directly proportional to its greatest stress; or, in other words, that in proportioning all members of trusses a constant intensity of working stress is to be used, while in reality for modern steel bridges the intensities often vary considerably in the same specifications. Again, no distinction is made between tension and compression members, and no account is taken of the greatly varying amounts of their percentages of weights of details.

There is, however, one mathematical investigation concerning economic truss depths which is approximately correct, and which is based on assumptions that are very nearly true; but it holds good only for trusses with parallel chords. It is this:

Let A = weight of the chords,
B = weight of the web,
C = weight of the truss,
and D = depth of the truss.

Then

$$C = A + B. \qquad \text{[Eq. 1]}$$

But the weight of the chords varies inversely as the depth, or $A = \frac{a}{D}$, and the weight of the web varies directly as the depth, or $B = bD$, where a and b are constants; and, therefore, $C = \frac{a}{D} + bD$.

If C is to be made a minimum, we shall have, by differentiation,

$$\frac{dC}{dD} = -\frac{a}{D^2} + b = 0, \qquad \text{[Eq. 2]}$$

or

$$-\frac{A}{D} + \frac{B}{D} = 0, \text{ or } A = B. \qquad \text{[Eq. 3]}$$

As the second differential coefficient, after substitution according to the usual method of maxima and minima, comes out positive, the result obtained corresponds to a minimum. From this it is evident that, for trusses with parallel chords, the greatest economy of material will prevail when the weight of the chords is equal to the weight of the web. The author has verified this conclusion by checking the weights of chords and webs in a number of finished designs, finding it to be absolutely reliable. However, it is not of much practical value, because the economic depths of trusses with parallel chords are pretty well known; and, again, when spans are in excess of 175 or 200 feet, the chords of through-bridges are seldom made parallel. Moreover, the best depth to use is not often the one which gives the least weight of metal in the trusses.

It has been found by experience that, for trusses with polygonal top chords, the economic depths, as far as weight of metal is concerned, are generally much greater than certain important conditions will permit to be used. For instance, especially in single-track, pin-connected bridges, after a certain truss depth is exceeded, the overturning effect of the wind-pressure is so great as to reduce the dead-load tension on the windward bottom chord to such an extent that the compression from the wind load carried by the lower lateral system causes reversion of stress, and such reversion eye-bars are not adapted to withstand. A very deep truss requires an expensive traveler, and decreasing the theoretically economic depth increases the weight but slightly; hence it is really economical to reduce the depth of both truss and traveler. Again, the total cost of a structure does not vary directly as the total weight of metal, for the reason that an increase in the sectional area of a piece adds nothing to the cost of its manufacture, and but little to the cost of erection; consequently it is only for raw material and freight that the expense is really augmented. Hence it is generally best to use truss depths considerably less than those which would require the minimum amount of metal. For parallel chords, the theoretically economic truss-depths vary from one-fifth of the span for spans of 100 feet to about one-sixth of the span for spans of 200 feet; but for modern single-track-railway through-bridges the least allowable truss-depth is about 30 feet, unless suspended floor-beams be used, a detail which very properly has gone out of fashion.

In two five-hundred-foot spans of a combined railway and highway bridge the author employed a truss depth of seventy-two feet; but this was determined by the reversal of stress in bottom chords through wind-

pressure. A greater depth, if permissible, would have caused a saving in total weight of metal. In another of his designs for a five-hundred-and sixty-foot span a truss depth of ninety feet was adopted, but in this case the live load was very great, varying from ten thousand pounds per lineal foot for short spans to eight thousand pounds per lineal foot for long ones; and the bridge is twenty per cent wider than in the case of the two five-hundred-foot spans just mentioned. The greater the live load and the wider the bridge, the greater generally can the truss depth be made advantageously.

The little mathematical investigation given in this chapter can be applied with fair accuracy to plate-girder bridges and to the floor systems of truss-bridges. If, for ordinary cases, in designing plate girders, one will adopt such a depth as will make the total weight of the web with its splice-plates and stiffening angles about equal to the weight of the flanges, he will obtain an economically designed girder, and a deep and stiff one. For long spans, however, this arrangement would make the girders so deep as to become clumsy and expensive to handle; consequently, when a span exceeds about forty feet, the amount of metal in the flanges should be a little greater than that in the web; and the more the span exceeds forty feet the greater should be the relative amount of metal in the flanges.

The true economic investigation for plate-girders is as follows, when the web is assumed to resist its share of the bending moment:

Let M = bending moment at mid-span,
h = depth of web,
t = thickness of web,
S = intensity of working stress for tension,
l = length of span,
and c = ratio of weight of details of web (i.e., end stiffeners, intermediate stiffeners, splice plates, and fillers) to weight of the web plate itself.

The sum of the two flange areas at mid-span, including an allowance of fifteen per cent for rivet holes, will be given by the equation,

$$F = 1.15\left(\frac{2M}{hS} - \frac{1}{4}ht\right); \qquad \text{[Eq. 4]}$$

and the total weight of metal in the flanges, taking into account the fact that the cover plates do not run the full length of the girder, will be given approximately by the equation,

$$W_f = 3.4 \times 1.15\left(\frac{2M}{hS} - \frac{1}{4}ht\right) \times 0.8\,l.$$

$$= 3.4\,l\left(\frac{1.84\,M}{hS} - 0.23\,ht\right). \qquad \text{[Eq. 5]}$$

The weight of the web and its details will be

$$W_w = 3.4\,l(ht + cht). \qquad \text{[Eq. 6]}$$

Therefore the total weight of girder will be

$$W_g = 3.4\,l\left(\frac{1.84\,M}{hS} - 0.23\,ht + ht + cht\right)$$

$$= 3.4\,l\left(\frac{1.84\,M}{hS} + 0.77\,ht + cht\right). \qquad \text{[Eq. 7]}$$

Differentiating with respect to h and placing the differential coefficient equal to zero gives

$$\frac{dW_g}{dh} = 3.4\,l\left(-\frac{1.84\,M}{h^2S} + 0.77\,t + ct\right) = 0. \qquad \text{[Eq. 8]}$$

Hence

$$\frac{1.84\,M}{hS} = 0.77\,ht + cht; \qquad \text{[Eq. 9]}$$

from which we find

$$\frac{1.84\,M}{hS} - 0.23\,ht = 0.54\,ht + cht, \qquad \text{[Eq. 10]}$$

and

$$3.4\,l\left(\frac{1.84\,M}{hS} - 0.23\,ht\right) = 3.4\,l(0.54\,ht + cht). \qquad \text{[Eq. 11]}$$

But the value of c is generally about 0.3. Substituting this gives

$$3.4\,l\left(\frac{1.84\,M}{hS} - 0.23\,ht\right) = 3.4\,l(0.84\,ht). \qquad \text{[Eq. 12]}$$

But the first member of this equation represents the weight of the flanges for the most economic condition, and the second member is eighty-four per cent of the total weight of the web plate without its details.

Dividing both sides of the last equation by 0.8 and canceling the 3.4 l gives

$$\left(\frac{2.3\,M}{hS} - 0.29\,ht\right) = 1.05\,ht, \qquad \text{[Eq. 13]}$$

or

$$1.15\left(\frac{2M}{hS} - 0.25\,ht\right) = 1.05\,ht. \qquad \text{[Eq. 14]}$$

Evidently the first member of this equation represents the gross area of the flanges and the second member differs only a little from the gross area of the web and may without any great error be called such. Hence it may be stated that the theoretical maximum of economy exists when the gross areas of flanges and of web at mid-span are equal—a condition readily remembered. If the depth of web be selected on this basis, rather than by the old criterion which makes the total weight of flanges equal to the total weight of web with all its details, it will be found to give a greater web depth. This increased depth is likely to augment the cost from one or more of the following practical considerations which the formula cannot take into account:

A. An additional splice or two in the web, or else a slightly increased pound price for the large plates.

B. Larger outstanding legs for all stiffening angles.

C. Reduction in the number of cover plates.

D. Narrowing of flange angles and necessitating thereby either an additional bracing frame or an increase in sectional area of the compression flange, in order to compensate for the greater ratio of unsupported length to width.

E. Possible thickening of web because of its greater depth.

F. Possible encroachment on under-clearance in deck spans, or raising of grade to avoid the same.

G. Possible difficulty in fabrication or shipment in case of long or heavy girders because of excessive depth.

Any one of these changes would be likely so to upset the economics of the case as to cause material decrease in the theoretical depth found by the preceding investigation. One will not often make an error in economy by following the old established rule in "De Pontibus" to the effect that the best practicable arrangement is generally to make the weight of the flanges equal to the weight of the web and its details; and there are occasionally cases where a saving of metal can be effected by making the web depth even smaller than that given by this old criterion, when by so doing a web splice may be avoided or smaller stiffening angles may be adopted. It should be borne in mind that there is quite a range in web depths over which the theoretic minimum weight is about constant, unless the thickness of the shallower web must be increased on account of the shear; hence one may often vary the dimensions of a plate-girder materially without affecting greatly the matter of economics. In Fig. 20*a* is given a diagram of economic depths of plate-girders with riveted end connections.

Concerning economic panel lengths for truss bridges, it is safe to make the following statement: Within the limit set by good judgment and one's inherent sense of fitness, the longer the panel the greater the economy of material in the superstructure. Of course, when one goes to such an extent as to use a thirty-foot panel in an ordinary single-track-railway bridge he exceeds the limits referred to, because the lateral diagonals become too long, and their inclination to the chords becomes too flat for rigidity. Again, an extremely long panel might sometimes cause the truss diagonals to have an unsightly appearance on account of their small inclination to the horizontal.

In plate-girder structures with floor-system of cross-girders and stringers, there is generally no economy in adopting long panels—in fact they are certain to involve an increase of total weight of metal; but, on the other hand, the cost of erection is probably lessened by reducing the number of field-driven rivets.

Warren trusses are cheaper than Pratt or Petit trusses for parallel chords, but not for those with a polygonal chord. The first-mentioned type

changes sectional areas of chords only one-half as often as do the others, which feature tends to save metal in splice plates and expense in field riveting.

The length of span at which it pays to change from parallel chords to curved or, more properly speaking, polygonal chords, will vary with the class of bridge; but it is seldom advisable to adopt the latter for spans under two hundred feet. The greater the panel length the greater the limit of span for parallel chords, consequently it will generally be found shorter for highway bridges than for railway bridges. This curving of the top chords of long through-spans has sometimes been carried to such excess

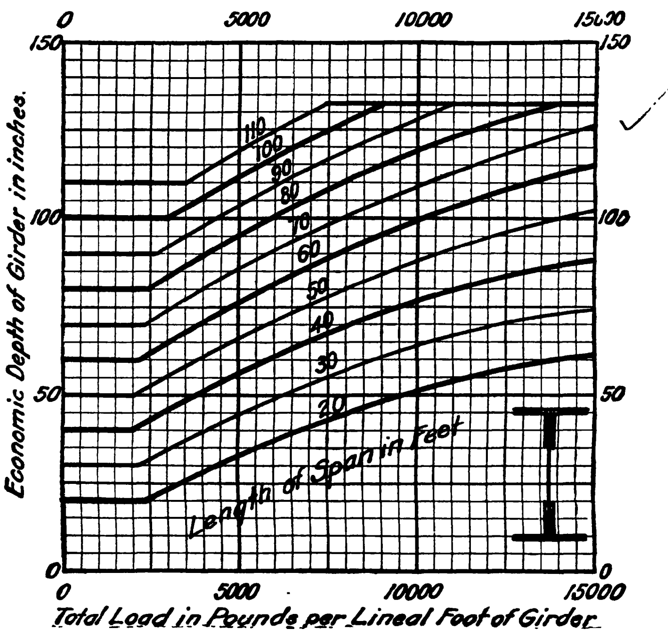


FIG. 20a. Economic Depths of Plate-Girders with Riveted End-Connections.

as to approach very closely the old parabolic trusses, in which the curve extends from end-pin to end-pin. In a large and important bridge over the Mississippi River the top chords of the main spans, which exceed five hundred feet in length, are so curved as to involve the use of a very shallow portal, allowing but the ordinary clear headway beneath. Such excessive curvature causes the top chord to do most of the work of the web and makes the latter too light and vibratory. It also necessitates the use of counters or stiff main diagonals almost up to the ends of the span. A proper curvature of the chords is not only economical of both metal and money, but also is æsthetic, adding greatly to the appearance of most bridges, consequently this feature should be encouraged, but not, of course, to excess. The best curvature of chords for any span can only be determined by experience,

the controlling factor being reversion of web stresses. In general, it may be said that the greater the arching the more artistic the effect. For highway bridges it can be made greater than for railway bridges, because the effect of impact is less in the former than in the latter; nevertheless, even in highway bridges the curvature must not be carried to excess on account of the tendency of light web members to set up vibration from insignificant moving loads.

CHAPTER XXI

ECONOMICS OF DECKS AND FLOOR-SYSTEMS

STEAM-RAILWAY BRIDGES

Decks

RAILWAY-BRIDGE decks may be of timber ties, either plain or treated, spaced from twelve to fifteen inches from center to center, with either two or four rows of wooden guard-rails bolted thereto; trough floors with or without ballast; ballasted road-bed resting on a solid floor of treated planking, steel plate, or reinforced-concrete slab; or rails resting directly on the steelwork.

The most common type, and the cheapest, as far as first cost is concerned, is the open, untreated-timber deck; but it may be more expensive than some of the other types when the item of maintenance is considered. It affords easy riding, but is not so safe against derailment or burning as decks that are closed. These advantages and disadvantages, however, cannot be evaluated in money. Treated ties, of course, cost more than the plain ones; but they generally last so much longer that they are more economical in the end. They can be of a cheaper grade of timber than the untreated ones, but this saving is offset by the necessity for using tie plates between the timber and the rails. Without these the life of the soft timber would be very short under heavy traffic. However, first-class construction calls for tie plates on all timber ties.

Trough floors with ballast and ties therein are uncommon. This was one of the first kinds of shallow floor to be built, but it has gone out of fashion. There was one tie per trough, and it generally rested directly on the steel; but sometimes a few inches of ballast were interposed. This type is noisy and expensive, and the replacement of ties in the trough is both difficult and costly.

Ballasted decks are more expensive in respect to first cost than the open ones, especially in long-span structures, on account of the augmented dead load. They provide easier riding; and for short-span bridges on first-class lines they are almost exclusively adopted as standard. One incidental advantage that they possess is that they permit the use of skew abutments, which are not compatible with the open-timber type of deck. Again, they are more conducive to maintenance of alignment; and they protect the steelwork fairly well from brine drippings.

The cheapest good type of ballasted floor is that in which the ballast rests on a solid base of creosoted planks; and a high authority on railroad bridge building and operation claims that, considering the cost of maintenance, this deck is cheaper than the open-timber one for spans of ordinary length. It would be very uneconomical to omit the treatment of the timber; because, in replacing the base, it is necessary to remove and store the ballast and later to put it back.

Ballast resting directly on steel plate is not much used, although it produces a shallow deck, which is sometimes a *sine qua non*. It is expensive, and the steel plate is liable to rust.

Ballast resting on a reinforced-concrete slab represents the highest grade of construction, but it is too heavy for long spans. In short spans, weight is desirable for high speed so as to check vibration, notwithstanding the fact that the cost of the steelwork is increased thereby. It is the best type for overhead crossings for the following reasons—the noise is reduced to a minimum, it can be built waterproof to exclude drippings, the depth can be made comparatively small, and the maintenance cost is the least practicable. This style of deck requires a specially-good drainage system; and, if the steel is encased, the deck must be waterproofed, which adds to the expense. True economy calls for a two-ply, three-ply, or even four-ply membrane of cotton drilling or burlap covered with asphalt of the proper consistency. The cotton has proved to be more durable than the burlap; and, therefore, it is the more economic. It is advisable for the sake of durability (and, consequently, for that of economy) to cover the waterproofing with either protection-bricks or thin concrete, the latter being the lighter and cheaper. An important portion of the work is the joint of the flashing with the web of the girder. This requires very careful attention in both design and construction, if leakage is to be prevented.

Rails laid directly on the steelwork give a minimum depth for the floor, but they require a rather expensive floor-system; and there being no cushion between rails and steel supports, the riding is not easy and the noise involved is excessive. For these reasons the type is not desirable; and the only excuse for using it is a compulsory call for an exceedingly shallow floor.

Spans without Floor-Systems

The floor-system proper may be omitted in I-beam spans, most deck plate-girder spans, those half-through plate-girder spans in which the wooden ties rest on either the bottom flanges of the main girders or on special shelf-angles, and short deck truss-spans in which the ties are supported directly by the top chords. This omission of the floor-system is entirely proper from the points of view of both economy and expediency, as far as I-beam and deck plate-girder spans are concerned, and even in the case of deck truss-spans when the truss-spacing is not too wide. Beyond the limit of ten feet from center to center of trusses the size required for ties

becomes too great for heavy loadings. It is true that this limit is sometimes made as high as thirteen feet; but then the ties are difficult to secure, and the maintenance cost thereof is large.

Resting ties on the bottom flanges or on special shelf-angles in half-through, plate-girder spans, although economic in first cost, is not good construction; because the removal and replacement of the said ties is abnormally troublesome and expensive. This practice, which was quite common two or three decades ago, is pseudo-economical, especially in view of the rapidly-augmenting prices of timber. Moreover, this detail has a tendency to distort the webs of the girders; and the depth available for cross-struts is small. These are needed for a proper staying of the top flanges of the main girders, and they have to be stiff in order to be effective.

Open timber decks can be used on I-beam spans, on deck, plate-girder spans, and on deck truss-spans without floor-systems, as can also ballasted decks supported by treated timber or reinforced-concrete slabs. In short I-beam spans the beams can be spaced closely and a thin slab can be employed, or the beams can be embedded in the slab, or there can be used longitudinal, reinforced-concrete troughs resting on the bottom flanges of the I-beams with the latter encased in concrete. For I-beam spans with either timber or ballasted deck, it is cheapest to adopt the minimum number of beams that will carry the load; but with limited headroom it will be found necessary to employ shallow beams and space them closely. If it be desirable for the sake of protection to encase the said beams in concrete, it is usually most economic to rest the slab on the beams and encase them separately; but for very thin floors it is cheapest either to embed the beams in the solid slab, or to use reinforced-concrete troughs.

Floor-Systems

The standard floor-system consists of two stringers per track riveted to the cross-girders, with the latter riveted in turn to the trusses or main girders. In general, this construction is by far the most economic type of floor-system; and it provides substantial floor-beams at panel points to serve as lateral struts. It can be used with any of the ordinary types of deck previously described. Auxiliary stringers, often termed "jack-stringers," are sometimes added so as to take care of derailed trains. Four carrying-stringers per track are occasionally employed, either because it is the policy of the road to do so or in order to permit the adoption of a shallow floor, especially in half-through, plate-girder spans. It is more economic to adopt four lines of carrying-stringers per track than to have only two of them and two lines of jack-stringers, for the reason that, except in the case of derailment, the latter are idlers. Four carrying-stringers, of course, require more metal than do two, especially with long panels; but with ballasted decks on concrete slabs they permit a reduction in the slab-thickness; and in spans of considerable length the reduction in dead load involved will nearly compensate for the extra stringer metal.

The economic panel length for this type of floor-system, as far as that floor-system alone is concerned, is always smaller than can be employed for single-track truss-bridges, whether through or deck, as can be seen by referring to the curves on pages 1224, 1229, and 1233 of "Bridge Engineering." This is also true for double-track, riveted-truss bridges, as shown by the curves on page 1239 of that treatise, although there is no great reduction below 25 feet. For double-track, pin-connected bridges, 25 feet is the economic length (see "Bridge Engineering," page 1244). This is due to the extra metal required for the cut-away ends of the floor-beams, making pin-span cross-girders much heavier than riveted-span ones. Where jack-stringers or four lines of carrying-stringers per track are employed, the economic panel lengths would be still shorter. It will be found, however, that variation in panel length will not affect greatly the total weight of metal in span, since lengthening the panels generally reduces the truss weight. For long, deep trusses, longer panels than usual give smaller total weights of truss metal.

In respect to the economic depths of stringers for two lines per track, they vary from 4 feet for 20-foot panels to 5 feet for 35-foot panels with light live loads, and from 5 feet for 20-foot panels to 6 feet for 35-foot panels for heavy live loads. The economic depths for four lines of stringers per track are about one foot less than the preceding.

The economic depths for floor-beams vary from 5 feet to 6 feet, according to panel length, in single-track bridges with light live loads, and from 6 feet to 7 feet with heavy live loads. The corresponding figures for double-track bridges are respectively, from 6 feet to 8 feet, and from 8 feet to 10 feet.

Under-clearance requirements often call for shallower floor-beams than those of economic depth, especially for double-track bridges. Very shallow, double-track, railway-bridge floor-beams are quite expensive. Any decrease in depth of floor-system below the economic one will increase the cost of main structure; but this is partially offset by the slightly-reduced lengths of approaches; consequently, when comparing the costs of floor-systems of different depths, this fact must not be forgotten.

Half-through, plate-girder spans are used only where the headroom is limited; hence the floor-systems for these structures are almost always shallow and uneconomic of metal. With open-timber decks, four lines of carrying-stringers per track are employed in order to provide a shallow floor; and the panels can be short and of economic length. Solid floors are quite common with through-plate-girder construction. The trough floor-system is very thin, and hence is applicable thereto; but it is expensive and otherwise objectionable, as hereinbefore explained. When a ballasted deck on creosoted timber is adopted, four stringers per track can be used, but it is more common to employ closely-spaced, transverse rolled I-beams, especially when the headroom beneath is small. The latter construction is also usually employed with a ballasted deck carried on a steel plate.

Ballasted decks on reinforced-concrete slabs are much used for half-through, plate-girder spans. The floor-system can be of the usual stringer-and-floor-beam type, generally with four lines of stringers per track, on account of restricted headroom; or the slabs can rest on rather-closely-spaced, transverse beams. With the latter arrangement it is economic of metal to adopt spacings of 5 feet or more; but, on account of restricted head room, it is frequently necessary to space much more closely, in order to permit the use of shallower beams. If the available depth is very small, these cross-beams can be embedded in the solid slab; or reinforced-concrete troughs resting on the bottom flanges of the beams can be employed, the tops of the beams being encased in concrete. The relative economics of the last three types depend on whether the steel needs to be encased so as to protect it from locomotive blast or other attack from below. If no encasement be required, the first type is the cheapest; but otherwise there is not much difference in the costs of the three. As before mentioned, in all cases where beams are embedded in concrete, the top surface of the slab must be water-proofed.

Rails resting directly on the steel require floor-systems of closely-spaced, transverse beams—an expensive construction and not good in case of derailment.

In respect to encasement, it is generally economic to put it on by cement-gun, because it is cheaper than the poured covering, and being thinner it saves in the dead load to be carried by the steelwork.

For double-track, deck truss-spans it is most economical for the floor-system to space the trusses 20 feet or 21 feet centers, and to use two lines of stringers about 7 feet centers, resting the ties on the stringers and top chords. Such a floor-system weighs but little more than that for a single-track through-bridge. It may take so much extra metal in the strengthened chords as to absorb most of the saving from the omission of the two lines of stringers, but the floor beams are much lighter than those ordinarily required for double-track bridges. There is also considerable economy involved in the lateral system and vertical sway bracing. In this layout of floor-system any of the three ordinary types of deck can be utilized.

In general it may be stated that for any span-length the floor-system of a deck span will be cheaper than that of a through span.

Stringers are usually plate-girders, except for through, plate-girder spans; and in these rolled I-beams are more applicable. Floor-beams are nearly always plate-girders, although occasionally in narrow structures it may pay to use deep rolled I-beams. Cross-beams in through, plate-girder spans having no stringers are nearly always rolled I-beams, unless the span be unusually wide.

In making economic comparisons between plate-girder and rolled I-beam constructions, it is necessary to take into account the difference in

the pound prices of the two types, the said difference usually, but not always, being in favor of the latter.

Electric Railway Bridges

This type of structure is found on interurban roads and elevated railways. The economic problems of the deck and floor-system are similar to those in steam-railway structures, except that the construction is much lighter. The economic lengths are somewhat greater because the loads are smaller, but the economic principles involved are identical.

Highway Bridges

Decks

For highway bridges there are two general types of deck, viz., timber and concrete. In respect to first cost of both the deck itself and the material required to support its weight, the timber type is always the cheaper, the saving increasing with the span length. In times past the timber deck *per se* was very much less costly than the concrete deck, but to-day it is otherwise, because the price of timber has risen more rapidly than that of any other material employed in bridge construction. Whether at the present time in any particular case timber or concrete *per se* for decks is the cheaper will depend somewhat upon the locality and the availability of supply for the various materials. The solution of the question has to be determined specially for each case, as it arises, but the difference in first cost for the two types will seldom be found great. The real economic question involved is one of comparative weights and costs of the materials required to carry the deck.

When, however, maintenance and renewals are taken into consideration, the concrete deck will nearly always be found to be the more economical for short spans and those of moderate length. There is, though, one factor of vital importance that cannot well be considered in the economic comparison, viz., the danger from fire. This is so important as to rule out the use of timber in bridge decks for all cases in which it is practicable to raise the money required for the concrete construction, excepting only that creosoted wooden blocks on a reinforced concrete base are permissible on account of being almost fireproof, or at any rate very slow-burning.

An exception might be made for the movable span in a bascule bridge because the tipping of the floor to a vertical, or nearly vertical, position is somewhat objectionable, although it is not impracticable to design a concrete floor with a concrete pavement thereon in such a manner as to withstand effectively the said tipping. In case the timber floor be adopted, it would be necessary to take the utmost precaution against injury by fire.

Pavings for Roadways

The type of paving to adopt depends upon whether any timber is to be used in the deck. Planks make the cheapest kind of flooring, but

they afford a poor riding surface and are short-lived. Transverse planks are objectionable on account of rough riding, and diagonal ones are better for high-speed traffic, although they splinter and are primarily somewhat more expensive—due to a waste in cutting them to proper lengths. Longitudinal planks afford the easiest riding, but they do not wear at all uniformly because of the tendency of the travel to run in certain lines.

Planks can be either untreated or creosoted. Creosoting delays the process of decay but lowers the resistance to abrasion; hence, for the wearing floor, untreated planks are the more economical. A hardwood timber that does not warp or twist excessively is the best for the said wearing floor. A double-plank floor with the lower layer creosoted and the upper layer placed diagonally is ultimately the most economical of all plank floors.

With the creosoted planks it is practicable to use a pavement of creosoted pine blocks, but the combination is very inflammable, and hence, is not truly economic. A similar pavement can be used to advantage for bascule spans by adopting instead of the blocks maple planks on edge bolted into groups and attached firmly to the steelwork.

With a concrete base any desired type of paving can be employed—wood blocks, brick, asphalt, bitulithic concrete or any other kind of bituminous paving, plain concrete, or granitoid. Wood block is the most expensive as far as first cost is concerned, but it makes a much better showing in the comparison when maintenance and renewal are considered. Brick *per se* is less expensive, but it is heavy and, in consequence, requires more metal to carry it. This is not a serious handicap on short-span bridges, but on long-span ones it is almost prohibitory.

Asphalt and bituminous pavements in general are good; and usually they are no heavier than the wooden-block ones. Unfortunately, they require an extensive plant to lay them; and, as the total area of paved surface on most bridges is comparatively small, the charge per square yard for use of plant will be excessive, unless there be a nearby plant available. To adopt an asphalt or bitulithic paving on a bridge in a small town is, for that reason, rarely economic practice. This difficulty, however, can be overcome by adopting an asphalt block pavement, which requires no plant for its construction.

A concrete wearing-surface in many cases is both satisfactory and comparatively inexpensive, for it requires no special plant to lay it; nevertheless an extra hard and durable aggregate is obligatory, and the concrete must be very carefully mixed, placed, and finished, and must be kept properly wet while curing, especially in hot, dry weather. Unless these precautions be observed, the concrete pavement will not prove economic because of short life and the expense of repairs and replacement. It will be found advisable to design with an allowance in dead load for an extra two inches of concrete, so that a thicker wearing surface may be put on, if ever desired, without overloading the floor-system or the trusses.

Sidewalks

Sidewalks are usually reinforced slabs of granitoid or specially-hard concrete, or else untreated planks. The latter are the cheapest, even when the cost of maintenance is considered, for they last fairly well; but, like all other timberwork, they are subject to destruction by fire, involving also injury to the metallic portion of the structure.

Sub-Paving or Base

As previously stated, the reinforced-concrete slab makes the best and most economic base for pavement. The reinforcing must be arranged to suit the steel floor-system, which will be described later. A timber base, either plain or creosoted is not truly economic construction because of the fire risk, nevertheless it is frequently used because of limited funds, or for lack of appreciation of true economy in design. Comparing creosoted and plain planking, the former is more expensive and heavier, but the plain plank as a base rots so rapidly that its use is uneconomic. Modern concentrated loads require that the distances between supports for the planks be small—say two feet for 3-inch planks and two and a half feet for 4-inch ones. The former thickness is totally unfit to support heavy traffic, especially after either abrasion or decay has started. With the live loads adopted in the standard bridge-specifications of today, timber joists or stringers are not truly economic; for very heavy members would be required for panels of any length, even down to the shortest ever likely to be considered. The cheapest timber floor is obtained by plank carried on ties supported by widely-spaced steel stringers. Common practice, however, spaces the stringers closer and puts nailing-strips thereon. This increases materially the cost of the stringers.

Curbs

The best kind of curb to adopt depends essentially upon the style of deck. With a concrete deck or a pavement on a reinforced concrete base, a concrete curb properly faced with a steel angle is the appropriate construction; but for timber decks, wooden guards are the proper thing. True economy demands that the drainage of the deck be correctly taken care of, either by openings beneath the curbs or by an effective grade on the roadway.

Waterproofing

When the steelwork is encased in concrete, or when no dripping of water on the space below the deck is allowable, it becomes necessary to provide waterproofing between the paving and the sub-base. The membrane type of waterproofing, previously described for railroad bridges, is the best and most economical.

Handrails

Handrails are made of concrete, structural steel, gaspipe, or timber. The concrete rails are the best looking, but are also the most expensive. Contrary to the general ideas of engineers, they will require repairs from time to time, in order to replace chips broken off from the sharp edges of columns by blows, or, by spalling in the case of concrete placed in cold weather and not thoroughly protected against freezing.

Steel handrails are expensive, when substantial; and they require painting from time to time.

Gas-pipe handrails are flimsy in appearance and ineffective besides.

Timber handrails are the cheapest, but like all other timber construction in bridges they are objectionable because of fire—besides, the ordinary ones are inherently ugly.

Electric-Railway Tracks

With timber decks the problem of caring for the electric-railway tracks is a simple one, but with a permanent deck it is somewhat difficult, involving, as it does, some economic considerations. In the first place, the rails must be of a height to suit the pavement adopted, and their heads must be flush with the top thereof. Next, the best method of support is a knotty point to solve. For a concrete deck there can be employed timber ties surrounded with either ballast or concrete, or steel ties embedded in concrete, or steel ties embedded in the reinforced-concrete slab and resting directly on the steel stringers, or steel rail-chairs supported in a similar manner. When the roadway and the railway are separated so that the two kinds of traffic cannot mingle, any of the types of floor previously described for steam railways can be used. The wooden ties are generally the least expensive type of track, and steel ties in a concrete base are costly. Steel ties in the reinforcing slab and resting directly on the steel stringers make good construction, especially for long-span bridges. Steel chairs are cheaper and fairly good, but they do not ensure a perfect spacing of the rails. Wooden ties in ballast are cheap *per se*, but the construction is heavy, and, therefore, expensive for all but very short spans.

Floor-Systems

The arrangement of the floor-system, *i.e.*, stringers, joists, floor-beams, and cross-girders, with their bracing, depends upon both the type of deck adopted and the kind of span employed.

In I-beam spans there is no need for a floor-system, because they are so short that a concrete deck is all they require; and its extra weight, as compared with that of the open deck of timber ties, does not add appreciably to the cost, because of the large ratio in any case of live load to dead load. The economical spacing of the I-beams varies from six to ten feet, increasing gradually with the span length. The under-clearance, however, may be

limited, requiring close spacing, so as to permit the adoption of shallower sections than the economic ones. The beams are frequently encased in concrete in order to protect the metal from the fumes of locomotives passing below. If a timber deck should be adopted with planks resting either directly on the stringers or on nailing-shims bolted thereto, the spacing should be some two and a half feet, more or less, depending upon the size of the concentrated wheel-loading; but, if the planks are carried on wooden ties, the spacing may be made about five feet.

In deck, plate-girder spans also, the floor-system can sometimes be omitted, especially in short spans, with either concrete deck or planks on ties; but in long spans of this type the steel floor-system will generally be found more economical. The limiting span-length for omission of floor-system is sixty or seventy feet, as far as the economics of the superstructure alone are concerned; but such extreme lengths may increase too greatly the cost of the supporting parts. For instance, with only two lines of main girders and a floor-system there will be required only two columns per bent and two pedestals to support them; whereas with several lines of girders there must be several columns and pedestals per bent, or else a heavy cross-girder or a continuous concrete pier, either of which is more expensive than a pair of columns with their pedestals and bracing. If for any reason it be decided to use continuous shafts in the piers, there may be no extra expense due to supporting several lines of girders; in fact, there may be a reduction in cost, because it might be possible to employ narrower piers or seats.

In deck, plate-girder spans with floor-systems it is economic to use only two lines of main girders up to a fifty-foot total width of deck or in some cases even sixty feet. The girders for economy should generally be placed at the quarter widths, so as to obtain, in both substructure and superstructure, the full economic benefit of the cantilevering. In narrow structures the spacing should be proportionately greater for the sake of rigidity and stability. With three lines of main girders, the spacing should be about one-third of the total width of the deck.

The standard floor-system in deck, plate-girder construction has stringers carried on floor-beams. The portions of the latter outside of the outer girders are cantilever beams with a strap-plate over the top of the main girder and some kind of properly designed detail below to carry the compression from the bottom flange of the cantilever to the bottom flange of the floor-beam. The stringers are generally rolled I-beams and the cross-girders built beams, although occasionally it will be found economic to employ a deep rolled-I-beam. With planks resting on stringers, either directly or by interposed nailing strips, the stringer spacing will, of course, be determined by the size of the specified concentrated live loads and the assumed thickness and kind of material of plank. It is generally about two feet for three-inch plank and two and a half feet for four-inch plank. Closer spacings than these involve an extravagant amount of metal for stringers.

With planks resting on ties, a spacing of five feet, or even more, can be used, depending, of course, on the size and strength of tie and the assumed concentrated live load. This wide spacing reduces materially the weight of metal in the stringers, thus saving more than enough to compensate for the cost of the ties.

With a concrete deck it is economical to space the stringers from six to ten feet, there being very little difference in total cost for variations between this range. It is better, as a rule, to adopt the smaller limit so as to reduce the cost of the slab-forms. The economic panel-length is small, running from ten or twelve feet for a twenty-foot spacing of main girders to fifteen feet for a thirty-foot spacing, and to eighteen feet for a fifty-foot spacing. It is specially short for closely-spaced stringers with plank floor. In general it may be said that the greatest economy obtains when one of the standard beams just barely figures as a stringer. For instance, if in choosing between a six-foot and a seven-foot stringer-spacing, the 20″ 65 lb. I just figures for the latter and must also be used for the former because the 18″ 60 lb. I is not strong enough, it will be found economical to adopt the greater spacing. On the other hand, if the 18″ 55 lb. I just figures for the six-foot spacing, and the 20″ 65 lb. I is required for the seven-foot spacing, the former will prove the more economic.

The question of extra metal in girders or trusses to carry heavier slabs is not important with short spans, but is very much so with long spans.

With concrete deck on plate-girder spans it will sometimes be found cheaper to use no stringers, but to employ rather-closely-spaced cross-girders on which the slab rests directly. The economic spacing of these girders figures out twelve feet or more, so far as quantities of materials are concerned; but it is better to use ten feet or even less, because the supporting of the slab-forms for the long panels is expensive. This type of floor is specially adapted to through, plate-girder spans and will be discussed further in connection therewith. It does not save quite so much with deck plate-girders, as the girders themselves form two or more lines of stringers; hence the total weight of stringer metal which can be saved is less with deck plate-girders than with through plate-girders. Again, an outside stringer may, in any case, be required to support the hand-railing.

When the structure carries electric railway tracks, a stringer must be placed under each rail. The addition of such stringers would practically eliminate the saving above discussed; and, therefore, the standard type of floor-system with stringers and floor-beams should be employed.

Sidewalk slabs should be carried on longitudinal stringers, even if the cross-girders are spaced closely. The outside stringer should generally be a channel so as to provide a flush surface for the attachment of the hand-railing; and with short panels the inner stringers may be of the same section. For five-foot walks two stringers will give the cheapest construction, the curb stringer of the roadway sometimes being utilized as one of

them. For eight-foot or ten-foot sidewalks three stringers are economic. Sidewalks on deck, plate-girder bridges reduce, or even eliminate, the economy of the stringerless type of floor-system.

With half-through, plate-girder spans, the positions and numbers of girders are generally fixed, two being just outside of the curb-lines; but in wide roadways it is economic to use three lines of girders, although the splitting of the traffic by the middle girder is not a desirable feature. In the case of a very shallow floor, though, such splitting cannot be avoided.

The stringerless type of floor-system is nearly always cheaper in half-through, plate-girder spans than the standard type with its stringers and floor-beams. With electric-railway tracks the advantage is smaller than where there are none. With no such tracks the saving of metal amounts to some 200 lbs. per lineal foot of span for a structure of any ordinary width; and the excess cost of the thicker slab is small. Again, the stringerless type is shallower than the other—which is often very important. By spacing the beams closely a very shallow floor can be obtained at moderate cost; but too shallow sections are not desirable, as they do not afford sufficient lateral support for the top flanges of the main girders.

The floor-systems of half-through, plate-girder bridges very often must be encased in concrete. This is to the advantage of the stringerless type, which has smaller area to cover and involves more simple work.

When a concrete slab is employed, it should be carried over to the webs of the girders and supported on shelf angles. If the floor is encased, the detail at the girder web needs careful attention, in order to prevent water from entering and thus causing rusting of metal and splitting off of encasement.

Sidewalks on half-through, plate-girder spans are carried on cantilever brackets, but outer steel stringers will be needed. The inner edge is supported on a shelf angle. A close spacing of these brackets is advantageous, as generally there is difficulty in properly taking care of large top-flange tension.

With truss spans, the standard floor-system of stringers and floor-beams is nearly always adopted, because it would usually be uneconomic to support the cross-girders by the chords between panel points. As previously indicated, the economic panel length for the floor-system *per se* is from fifteen to seventeen feet, which is too short for the economics of the trusses, hence longer panels than that limit generally have to be adopted. For a bridge of practically any width of roadway, the metal in the floor-system with 25-foot panels weighs about 70 pounds per lineal foot more than that required for the economic panel length, and for 30-foot panels about 150 pounds per lineal foot more. Divided panels in spans of moderate length permit the use of the economic panel length, but the secondary truss-members increase the weight of metal in trusses enough, or more than enough, to offset the saving in the floor-system. With long spans, however, this uneconomic feature disappears.

In timber decks the economic stringer spacing is about the same as it is in plate-girder spans; and in concrete decks it is five or six feet for spans of 200 feet or under, and less for longer spans. For instance, in spans of 400 feet with 25-foot panels it is four feet, and with 35-foot panels it is four and a half feet.

The stringerless type of floor system with concrete deck for very short panels—say 12 feet—shows the same economy in truss spans as it does in half-through, plate-girder spans. With truss-spans of moderate length, the panels are so long that the chords would have to be built as girders in order to carry cross-girders between panel points. The floor-system itself, as compared with that of the stringer type, when the panels are 25 or 30 feet long may show for standard deck-widths a saving of from 250 lbs. to 350 lbs., per lineal foot, while the extra metal in the chords required to make them serve as girders would amount to 250 or 300 lbs.; besides which there must be taken into consideration the increase in weight of truss-metal due to the augmented dead load by reason of the thicker slab. On this account the stringer type will nearly always be found cheaper, especially when electric railways are carried. With encasement there may be an economic advantage in the stringerless type, especially if a very shallow floor be called for. With extremely long panels it will be found cheaper to use stringers close to the trusses rather than to stiffen the chords of the latter enough to make them able properly to carry the transverse loading.

For very long spans with concrete slabs it is desirable to reduce the weight of deck to a minimum. In such cases it will be most economic to support the slab on closely-spaced cross-beams carried on stringers rather widely spaced—say 10 or 12 feet centers. Considering both the weight of metal in the floor-system and that in the trusses, it is found that the economic spacing of the small beams is about three feet. Comparing this with the ordinary type having stringers spaced four and a half or five feet, it will be found that the metal in the cross-beam type weighs about three pounds more per square foot for 25-foot panels and about one pound more per square foot for 35-foot panels, while the slab weighs eight pounds per square foot less. The total deck, therefore, weighs some five pounds per square foot less for 25-foot panels and seven pounds per square foot less for 35-foot panels. Considering the extra weight of truss-metal needed to support this excess of dead load, and remembering that the pound cost for the cross beams is a little less than that for the remainder of the floor-system, it has been figured that the cross-beam type is cheaper with 35-foot panels for spans in excess of 200 feet, cheaper with 30-foot panels for spans of 350 feet, and cheaper with 25-foot panels for spans of 600 feet. Evidently, therefore, there is no economy in adopting this type for spans shorter than 250 or 300 feet; but for all long spans it is quite economical.

This type of floor affords a good support for the rails, which must,

however, be made quite deep. The reinforced slab must be cut back from the rail in order to permit of the latter being removed. This detail requires careful watching.

Sidewalks on through-truss bridges are usually placed outside of the trusses and carried on longitudinal stringers supported by cantilever brackets.

In very-long-span bridges, such as cantilever or suspension structures, the matter of economy from deck and floor-system should be very closely studied, in order to eliminate every unnecessary pound of dead load. Preliminary designs should be made for various types and for different spacings of beams and stringers, in order to determine the most economic arrangement. The question of total weight of deck may be far more important for such long-span structures than the total weight of metal in the floor. Timber decks, of course, will be much cheaper than concrete decks; and the decision between the two types must be made by judgment, considering the nature of the traffic, the money available, the possible revenues, the first cost, the cost of maintenance, and especially the danger from fire. In general it may be said that the concrete deck should be adopted if sufficient money can be secured, and, for a toll bridge, if the estimated revenue justifies the expenditure.

There is a light type of floor recommended for long spans by Edward A. Byrne, Esq., Member American Society Civil Engineers, Chief Engineer of the Department of Plant and Structures of the City of New York. It consists of stiffened buckle plate, having the buckle down, covered with plain concrete, and supporting a thin block pavement. Concerning this type Mr. Byrne on June 8, 1920, wrote the author as follows:

> Where buckle plates are used to support the roadway pavement the following detail of construction is suggested. The ends of all buckle plates should rest on supports and be properly spliced. The fillets of the buckle plates should be reinforced by angles —$3'' \times 3'' \times \frac{3}{8}''$ angles have proven effective in buckle plates supported on stringers five feet on centers. The plates should be laid with the buckle down and filled with Portland cement concrete to a depth of at least three inches above the top of the plate. Longitudinal angles, $3'' \times 3'' \times \frac{3}{8}''$, should be riveted to the buckle plates along the line of the supporting stringers to restrain the concrete foundation and act as a template for laying the concrete. Each buckle plate should have a drain hole; and a hole in the concrete, extending from the top thereof to the buckle plate, should be provided. On top of the concrete foundation a wood block pavement, three inches in depth, should be laid without any cushion, sand to be spread over the pavement and brushed into the joints. The Portland cement concrete should be one part cement, two parts sand, and four parts broken stone or gravel—the top to be rubbed to a smooth finish. . . .
>
> It is highly important that, in using a buckle plate floor, all deflections of plates be eliminated, otherwise the concrete foundation will crack and disintegrate.

Tests made by Mr. Byrne on the floor of the Queensboro Bridge, where the stringers were spaced five feet on centers and where the buckles were about four feet long, showed deflections of buckle plates as great as three-eighths of an inch before the stiffening angles were attached, but no appreciable amount after they were put on.

The cost of the pavement-support *per se* is about twice as great for the buckle-plate type as for the ordinary type with reinforced-concrete slab,

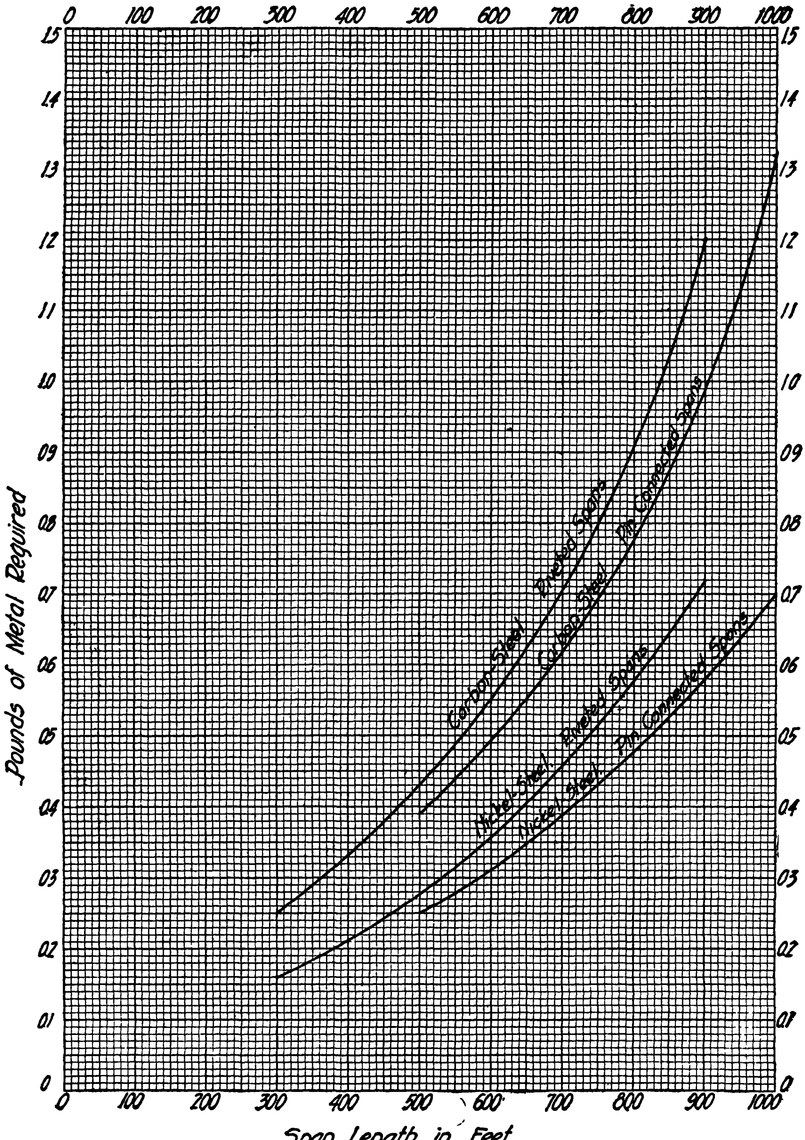


FIG. 21*a*. Diagram of Increase in Weight of Metal for Each Excess Pound of Extraneous, Uniformly-Distributed Load in Simple-Truss Spans.

the difference at current prices of materials and labor being about eighty cents per square foot of floor; but there is a saving effected in the dead

load by the former, amounting to about eighteen pounds per square foot. For a simple truss span of 600 feet, it requires about 0.58 lb. of steel to

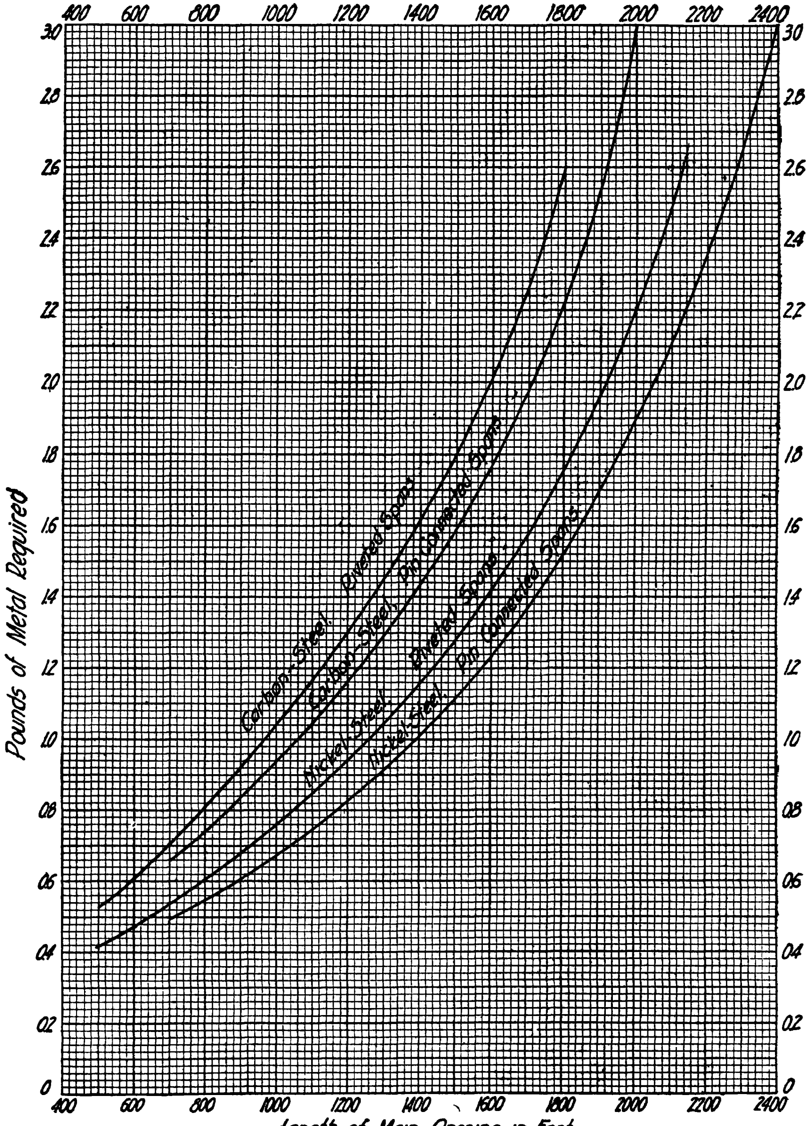


FIG. 21*b*. Diagram of Increase in Weight of Metal for Each Excess Pound of Extraneous, Uniformly-Distributed Load in Type-A-Cantilever Bridges.

carry an extra pound of extraneous load; hence, for such a span, the extra metal to support the eighteen pounds of excess dead load of the floor would

be about ten and a half pounds, which, at the present unit prices of material and labor, are worth in place about 84 cents, showing that to-day for simple-truss spans exceeding 600 feet in length it would be economic to adopt the buckle-plate floor.

As this economic question is likely to arise at any time, and as its solution is largely dependent upon the relative unit prices of concrete and structural steel in place, the author has prepared the diagrams shown in Figs. 21*a* and 21*b*, which indicate for simple-truss and cantilever spans, respectively, and for both carbon steel and nickel steel, the amount of extra metal that will be required to support one excess pound of extraneous load uniformly distributed.

Just as the MS. of this book was about to go to press, the author's attention was called to a light aggregate for concrete, termed Haydite, being named after its discoverer, Mr. Stephen Hayde, a well-known citizen of Kansas City, Mo. He claims that concrete made of it weighs only 108 lbs. per cubic foot, as against 150 lbs. per cubic foot for ordinary 1 : 2 : 4 concrete of small broken stone or gravel, and that the lighter concrete not only is practically impervious to water, but also that it gives some 25% greater resistance to compression than its heavier competitor. If these claims are correct, there should be quite a demand for Haydite for the bases of highway-bridge floors, especially as the extra cost of the aggregate is moderate. Mr. Hayde and his business associates contemplate retaining the author to make a thorough investigation of the suitability of Haydite for pavement bases in bridge construction and to show by diagrams and otherwise the economics involved thereby. Until some engineer of standing tests the new aggregate and vouches in print for its characteristics, the profession will have to take the discoverer's claim *cum grano salis*; nevertheless, the author has great hope of its being proved to be all that is claimed. If such be the case, its use will result in a real boon to the builders of modern, first-class, highway bridges, especially those of long span.

CHAPTER XXII

GENERAL ECONOMICS OF DESIGNING AND DETAILING

In this chapter will be treated only the salient economic features of ordinary designing and detailing, because to attempt to do more would lengthen this portion of the book beyond all reason.

In all cases it is economic to choose the most simple types of structures and details, and especially those that lend themselves best to stress analysis.

One should select members which by form and location are best adapted to resist forces economically and to carry stresses by shortest routes. For instance, in a long-span truss with parallel chords, shear is transferred entirely by the web members; and the path of the web stresses is much shorter by the divided-triangular truss than by the Pratt truss, hence the use of the former is economical. But in short-span trusses the verticals are nearly all of minimum section, hence there is but little, if any, difference in the economics of the two types. Similarly, with polygonal top chords, the shear on the webs being comparatively small, there is not much to choose between the said two types.

In riveted tension members the riveting should be arranged so as to reduce to a minimum the number of rivet-holes to be taken out of a section; but if the number assumed be too small, material will be wasted at splices in developing the net section, because extra holes above the number that would normally be employed cannot be used at joints unless extra section is first developed into the splice-plates or gussets. Furthermore, the section will very likely be reduced in detailing, without the fact being noted. This will overstress the metal at such sections.

In compression members the metal of the section should be arranged to secure the largest practicable radii of gyration without involving the use of metal that by its thinness would transgress the rules of standard specifications, viz., minimum limits of one thirty-second of unsupported width for webs, and one fortieth thereof for cover-plates. It is economic to employ fairly heavy angles, as thereby the radii of gyration are increased and, in consequence, also the strength of the strut. If the unbraced length of the piece is greater in one direction than in the other, the section should be so arranged that the values of the ratios of length to radius of gyration are approximately equal. For instance, in the case of intersecting diagonals of lower lateral bracing, the unbraced length in one direction is twice as great as that in the other. Then for 2-angle sections unequal-legged

angles should be employed with the longer legs vertical; but for 4-angle sections it is economic to make the longer legs horizontal and connect the shorter legs by a single line of lacing placed between them. For very long struts it is economic to employ four angles in box section, connected by four lines of lacing.

In light bridges the posts can be made either of H section or of four angles with either a web or lacing between, rather than of two channels laced. This arrangement saves considerably in weight of details, but some of that saving is lost because of the greater sectional area required to allow for the reduced radii of gyration.

For compression members which carry shear, there should be one or more webs in planes parallel to the said shear. Such webs can be employed to advantage also in simple struts of large cross-section. If lacing can be omitted thereby, a saving of metal may result. On the other hand, the radius of gyration may be reduced by using such diaphragms, thus lowering the unit stresses.

In railway, deck plate-girders, as far as weight of metal is concerned, it is economical to use cover plates for top flanges, but this requires rivets and the variable dapping of the ties. Rivets increase the work of placing and maintaining the ties, and deep dapping is undesirable. The best railroad practice forbids the use of rivets through the upper horizontal legs of the flange angles of stringers when timber deck is employed.

Batten plates for large compression members must be excessively thick, unless they be properly stiffened by angles; and it will generally be found less expensive to adopt thin plates and stiffen them. Unless considerable metal be saved by that expedient, however, it is better not to do this, because the addition of stiffening angles increases materially the amount and cost of the shopwork—besides it may involve some difficulty in handling the members.

In compression members consisting of two built or rolled channels, it is usually economical of metal to turn the said channels in. However, this increases the cost of the shopwork, because lacing-rivets when the channels are turned in usually cannot well be machine driven.

In designing a built chord for a bridge, the sections must be determined by considering all panels of the chord simultaneously. It is well to adopt as few differing thicknesses of metal as practicable, and to arrange the various sections so as to avoid, to as great an extent as possible, the use of filling plates at the splices. In field splicing the loss of loose filling plates is a cause of serious trouble and expense; hence they should be avoided. For the sake of simplicity, it is frequently best to waste metal in some of the panels; but such waste should always be reduced to a minimum. When cover plates are employed, it is advisable to avoid changes in the location of the center of gravity of the section.

In detailing one should always endeavor to use metal to best advantage; and the strength of each detail or connection should be figured whenever

this is practicable. When exact analysis is impossible, an ample amount of metal should be used so as to avoid any possible weakness due to such uncertainty; but this does not mean that it is well to be recklessly extravagant of metal in order to save the trouble of figuring the strength.

It is false economy of the worst type to skimp details; because the saving in metal is comparatively small, and the loss of strength may be very great. It is worse than useless to adopt low unit stresses for the main members and put in weak details; for the stress sheet then gives a false sense of security. It would be much better to have strong details and high working stresses in the main members.

Details and joints frequently defy exact analysis; and in such cases the designer should not fail to make approximate analyses to determine the character and magnitude of stress in every part so as to avoid all possibility of the existence of weak spots. It is not unusual to see metal placed at a joint where it could not possibly do any good, while an important component part of a member is left unspliced. A ten per cent error in strength on the side of danger will rarely do any harm; but the complete omission of a vital part of a detail may be very serious. For instance, in the lateral systems of highway bridges, it is not uncommon to see a diagonal of strong section connected by details so flimsy that they could not possibly transmit one-quarter of the stress which the said diagonal is capable of withstanding. The metal in such a lateral system is nearly all wasted.

As an important matter of economy, rusting and other kinds of deterioration should be carefully considered when making designs. It is very uneconomical to use parts which will rust out or wear out in a small fraction of the possible life of the structure. Repairs are not only expensive *per se*, but also they generally interfere seriously with the traffic. Some important railroads use $\frac{7}{16}$ inch or even $\frac{1}{2}$ inch as the minimum thickness of metal in steel superstructures; because they find thin metal to be one of the most fruitful causes of replacement. The author, however, is of the opinion that, if the said companies were to adhere to the $\frac{3}{8}$ inch minimum thickness and keep the metal always properly painted, they would obtain more satisfactory results.

All parts should be easily accessible to the paint brush; for otherwise the painters will fail to cover some metal that is difficult to reach, thus curtailing the life of the structure. A horizontal plate in a bottom chord will frequently save metal; but it invites rusting, and therefore is sure ultimately to prove uneconomical.

When there is a choice of plates and shapes that can be used in the make-up of a member, it is sometimes practicable to economize a little by adopting the most inexpensive ones; but too often it will be found that the more expensive shapes are also the ones which are the more appropriate and serviceable.

CHAPTER XXIII

ECONOMICS IN DESIGN FOR SHOP CONSIDERATIONS

While the consulting bridge engineers of America do not yet agree entirely with the engineers of the bridge manufacturing companies concerning all points of design, the ideas of both have of late been gradually getting closer together. The differences of opinion are generally in the line of economics, the shopmen desiring to cheapen work in ways of which the consulting engineers disapprove.

Designs should be made so as to afford the bridge shops every facility possible for using their machinery to advantage. For instance, details should be arranged for multiple-punch spacing, and to suit the requirements for bending, machining, and the various other operations which are governed by the shop equipment.

One great bone of contention in times past was the matter of sub-punching and reaming as against punching full-size and running a loose reamer through the holes in the assembled component parts, so as to ensure the possibility of the passage of the hot rivets. Even today much work is done by the latter method, but it is generally confined to parts of so-called minor importance. The object of sub-punching and reaming is two-fold: first, to make the holes in the component parts match properly after the passage through them of rigid-drill reamers, and, second, to cut away the metal around the peripheries of the holes which was injured by the brutal process of punching. The author, if he could always have his way in this matter, would bar entirely the punching of holes full-size; and he believes the day is coming when even the sub-punching and reaming will be prohibited by adopting the method of solid drilling. He has asked some of the prominent bridge manufacturers how much extra it would cost to employ the latter method, and has been told that, if the shops were properly outfitted for the work, the excess cost as compared with sub-punching and reaming would be practically *nil.* For nickel steel and other high-alloy steels solid drilling should be exclusively employed.

In the case of sheared edges, if the metal near the shear is to be depended on for strength, the said edges should be planed, but otherwise the planing should be omitted, unless the raw edges would be too much in evidence to the beholder of the finished structure. It is more than probable that the shearing of edges is just as destructive as the punching; for the brutality of the treatment of the metal in the two cases is of the same character and apparently of the same severity.

While it is undoubtedly difficult to procure a tight fit for stiffeners on rolled I-beams, it does not appear to the author safe to omit them at the ends of railway girders that are supported from beneath, or even from those used for carrying heavy highway loads to the masonry, because the unsupported webs are not of a shape satisfactorily to resist severe pounding.

Although it is true that turning the flanges of channels in makes the riveting somewhat more difficult, it need not prevent the use of power riveters, except in the case of a few rivets; while it facilitates greatly the detailing by bringing all the webs of main truss members in the same plane for the attachment of the gusset plates. Most of the author's riveted bridges are built in this way.

Again, it is important to have the batten plates inside of the gussets, and to carry them to near the ends of the members, both of which conditions the turned-in channels permit. Moreover, they generally involve an economy of weight of metal for lacing and battens. But one of the most important advantages of turned-in channels is that they permit the ends of the floor-beams to fit closely to the bottom chords without cutting either the chord or the beam, which is not practicable if the flanges of the bottom chords turn out.

In viaduct construction some manufacturers use their influence to have the girder-depth the same from end to end of structure, which is uneconomic of material, because the tower spans and the intermediate spans are nearly always of different lengths, which arrangement would call for different depths unless metal is to be wasted. The manufacturers' claim has apparently some justification if the tops of the columns are cut off so as to let the main girders be supported directly thereon; but, in the author's practice, the columns are carried up to the level of the deck, and both the longitudinal girders and the tower cross-girders abut into them; hence there is no valid objection to making the comparatively-short longitudinal-girders over the towers shallower than the long, intermediate longitudinal-girders. This layout certainly looks much better; and the corner brackets afford an excellent connection for the diagonals of the longitudinal bracing.

If the designing of details be left to the manufacturers of the metalwork, they often place the end or pedestal pin of a riveted-truss span below the bottom chord, forgetting that the thrust of a braked train, acting with a lever arm equal to the vertical distance between the center of the chord and the center of the pin, produces a large bending moment that has to be resisted by the stiffness of the bottom chord and that of the inclined end post.

When bridge superstructures are let to the manufacturers by the lump sum and they have the designing to do, they like to omit the end floor beams of through bridges, substitute instead inexpensive struts, and rest the stringers on the masonry; but the author believes that invariably end

floor-beams should be employed and be riveted to the end posts of the trusses so as to make the lower lateral system a harmonious whole; and it matters not if the end posts be inclined.

Manufacturers are willing to use single angles in tension, but this is objectionable because of the violation of the rules of symmetry and the consequent causing of secondary stresses.

A mooted point in designing is the exact location of top-chord pins. The author believes they should always be placed either on or a very little below the gravity lines of the sections, for it takes an exceedingly small eccentricity to produce a high intensity of bending stress on the chord. It is not right to assume that the reverse bending moment due to the weight of the member between panel-points will entirely counteract the bending moment due to the eccentricity; because the form taken under loading by the center line of the long strut will be a waved line passing through the centers of the chord pins, being concave upward in one panel and convex upward in the adjoining one. The amount to lower the chord pins below the centers of gravity of the sections is to be determined by making the compressive intensity due to eccentricity equal to the tensional intensity due to bending from weight of member. In any case this adjustment is a matter of compromise on account of the shifting of centers of gravity from centers of figure by reason of the variation in make-up of section from panel to panel, the amount ordinarily varying from zero to a quarter or three-eighths of an inch. In large bridges, of course, the variation will be greater than this.

Manufacturers like to use cast iron in bridges on account of its comparative cheapness per pound; but on general principles the author tries to bar out all cast iron from his bridges, fearing that, if it be permitted in one place, the contractor will insist upon putting it into another. Cast iron is nearly always inferior to cast steel for any purpose.

There is an uneconomic detail which is too often employed in both trestles and elevated railroads, viz., the insertion of a heavy casting between the lower end of a column and the masonry. The author has never been able to perceive the philosophy of this detail; for it involves the planing of a large amount of extra surface as well as a considerable increase in the weight of metal. Moreover, the additional surfaces in contact do not militate towards rigidity, for perfect contact is not always attained. The object is evidently the spreading of the load over the masonry; but this could be accomplished just as effectively and at less cost by using a rolled base-plate of the proper size and carrying the load to it from the column section by means of vertical plates, horizontal connecting angles, and vertical stiffening angles. The necessity for giving the latter a tight fit at their lower ends requires some troublesome shopwork, but the additional cost thereof will not offset the expense of the extra amount of planing involved by the casting detail.

In respect to the general and detail principles of the economics of shop-

work advocated by the highest authorities on the manufacture of structural steel and concurred in by the author, the following may be stated.*

Attention should be paid by designers to the different pound prices for the various sections, and they should remember that these variations are likely to change from time to time. For instance, all angles over 6″ and all beams over 15″ deep cost a small amount above the base price; and for large plates the extras increase with the widths by a rapidly augmenting scale, starting generally from 100″ with a trifling amount and reaching as much as one cent per pound extra for a width of 130″. It is, therefore, often more economic to adopt the shallower of two widths and use a little more metal, especially when the variation of an inch or two of girder-depth would change the pound price of the raw material in the web plates as much as a quarter of a cent. Large differences have existed at times during the past few years, plates sometimes being very expensive and almost unprocurable.

In the design for structural work for all purposes, more consideration should be given by the designer to the sections which are employed. Special material should be avoided, if possible; sections varying by $\frac{1}{16}$ inch should be so combined as to use one section as far as practicable; and special sections in small quantities should be eliminated entirely. Very often the delivery on the contract is delayed because the shop has to wait for a small quantity of a special section which is not rolled on time. Compliance with the above will insure better deliveries from the mill and quicker fabrication in the shop; and all parties concerned will be benefited thereby.

When ordering plates, the designer should adhere to standard dimensions as far as possible. This can always be done in the case of lateral and gusset plates, but a special depth may be necessary at times for the webs of stringers or girders. Standard widths for plates are 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 18, 20, 24, 30, 36, and 48 inches.

Eye-bars, adjustable members, turnbuckles, screw-threads, segmental rollers, clevices, upsets, etc., should be designed according to the standards of the bridge manufacturers, because quicker deliveries and better fabrication are obtained by the use of standards.

Designs should be made so that all extra or unnecessary operations in the shop are avoided. This should apply particularly to large and heavy members and to small members used in large numbers. The work on these pieces should be kept as simple as possible. When there is an extensive duplication of any piece, it will pay in its designing to save every pound of metal that can legitimately be omitted.

Reaming to templets is useless unless the templets can be set from finished surfaces, as in chord splices, ends of stringers, floor-beam connections, etc. Reaming of laterals to templets is liable to do more harm

* The data for most of the remainder of this chapter were furnished to the author in 1915 for Chapter XVII of "Bridge Engineering" through the courtesy of Messrs. Paul L. Wolfel and Albert F. Reichmann.

than good, as no finished surfaces are available. The same applies to diagonals in trusswork. While in a punched connection a few holes may be slightly out, which can be corrected in the field, if a connection is reamed to templet and the templet is not properly set, all holes will be equally out. Riveted trusses should be reamed and match-marked in the maker's shop when assembled.

In all plate girders and truss-bridge stringers the lateral system should be dropped so that the rivet heads thereof will clear the ties.

Wherever possible in heavy work, avoid, in the construction of the chords or web members, side plates or doubling up of the web plates. It will often pay to use heavier web plates without side plates, even if they have to be drilled from the solid. If, however, webs have to be doubled up or side plates used, the stitch rivets made necessary by this construction should be reduced to a reasonable amount. If a plate is used as a cover plate in a chord, it is good practice to limit its thickness to $\frac{1}{40}$th of the distance between rivets. If the same plate were used as a side plate in a chord, in most designs two or three times as many lines of rivets would be called for as would be necessary by the above limits.

It is cheaper and better to use heavy flange angles in stringers than lighter angles with cover plates, even if the said angles should have to be drilled from the solid.

Beveled cuts are to be avoided whenever possible, especially beveled cuts for angles that cannot be obtained by cutting multiple pieces from a long piece; also beveled cuts in all beams and channels, as these have to be sawed.

One of the greatest savings in recent years in bridge shops has been made by the use of multiple punches. These not only reduce the cost of the punching proper, but also save the cost of making templets and the laying out of the material. They further give far superior work; as the effect of the stretch of the material during punching on the accuracy of the work is eliminated, if these multiple punches are properly constructed. Their use, therefore, should be encouraged in every way. In order to do this, it is necessary to:

(*a*) Keep all rivets in line longitudinally.

(*b*) Keep as many rivets in line transversely as possible and do not use any more combinations of rivets transversely than necessary.

(*c*) Never have the longitudinal lines of rivets less than $2\frac{1}{4}''$ apart, nor the transverse lines less than $1\frac{1}{4}''$.

Do not crimp stiffeners if it can be helped, especially do not crimp stiffeners of short lengths, say up to about three feet. If stiffeners are crimped $\frac{3}{4}''$ or more, the crimp is unsightly; and better and more sightly work will be obtained by using a thin filler with a smaller crimp. Do not call for fillers or splice plates to have a tight fit, as this is impracticable in the shop. The stiffeners, of course, should have a close bearing.

Do not call for planing of the base, cap, sole, or masonry plates, as the mills can roll the same closer than they can be planed.

The old rule of sixty degrees for single lacing should be abolished, because this makes the lacing entirely too close for narrow members, and it is quite expensive and unsightly. Lacing bars should generally be lapped, as this detail will save about half the rivets. Instead of lacing it is often advisable to employ a solid web. This will sometimes permit the use of lighter main angles by counting the web as part of the section, although it reduces the radius of gyration and consequently increases the sectional area of a compression member — besides, it greatly facilitates the painting.

All hand riveting should be avoided wherever possible, also all odd riveting that has to be done either before the work is assembled or after a piece is otherwise finished.

In short plate-girder-spans it is economic to omit the bottom lateral system. Lug angles on laterals are expensive—it is better to use larger connecting plates in order to get the requisite number of connecting rivets.

In laying out viaducts, as many towers as possible should be made alike by varying the heights of the pedestals.

In square-girder spans, the number of panels of bracing should be even, but in skew-girder spans it should be odd. The greatest amount of duplication in any skew-span will be obtained if the floor is laid out so that the entire span can be revolved around a central point.

In pin-connected work the sizes of the pins should not be varied more than is strictly necessary, two or three sizes being generally sufficient for one span. It is not altogether a waste of material to use larger pins than necessary, as the bearing plates can be reduced in thickness and often in length.

It is economic of shopwork to avoid running the top flanges of stringers over the tops of the cross-girders.

In riveted tension members it is well to use tie plates instead of lacing. The former have the advantage of getting better shop rivets.

It is not necessary that the web-plate of a plate girder should be in contact with the sole plate, and to make it so is expensive.

In deep girders the web thickness should not be less than $\frac{1}{180}$ of the depth thereof, as otherwise buckling is liable to occur.

Where webs are spliced, ample clearance should be allowed, and the depth of the web should be one-half inch less than the distance from out to out of flange angles.

Rounded corners in plate girders are expensive, but sometimes they are required for the sake of appearance. Cambering of plate-girders is useless and quite expensive.

Information should be furnished to the fabricating shop, specifying the end of the structure which is to be erected first, it being very desirable to fabricate the work in the order of erection and also to note the direction of long plate-girders, so as to save turning them during shipment, at the shop, or in the field.

On chord or column sections extending over two panels with the same depth of section, but with smaller area required, the increased weight of the shop splices will tend to offset the increase in weight due to making both sections the same, the big advantage in the latter construction, of course, being that the material is continuous without the splice.

Frequently, on stringers and light girders, the webs are designed very light, which necessitates the use of many stiffeners to prevent buckling. It is often a big advantage to thicken the web and omit the stiffeners. The weight in either case is about the same, as the omission of the stiffeners will partially offset the increased weight of the thicker web.

For chord sections, the employment of reinforcing plates between angles should be avoided by using additional web-plates the full depth of the chord. This design has the advantage of connecting more of the main material to the flange angles direct, and avoids the use of a great many rivets which are necessary to connect the reinforcing plates to webs. When two webs are riveted together, the rivets should be about 12″ from center to center, the edges of the webs, of course, being held together by the rivets through the flange angles.

When the specifications call for material drilled from the solid on account of the use of either alloy steel or very thick ordinary steel, the members should be designed with as few pieces as possible. Instead of using $\frac{5}{8}''$ or $\frac{3}{4}''$ plates, which generally are of the right thickness for punched work, the material should be ordered as thick as permissible within the mill requirements, provided that the strength of the plates does not drop below the specification stipulations on account of insufficient rolling.

The preceding portion of this chapter applies specially to the practice of the consulting engineer, although it records the opinions of experts in the line of steel manufacture; but the following pertains specially to the drafting work in the office of a bridge manufacturing company. Its inclusion is in the nature of an afterthought; and the explanation thereof is as follows:

Long after the supposed finishing of the chapter, the author was being shown through the shops of the American Bridge Company at Ambridge by an old friend of his, Mr. C. M. Canady, one of the principal engineers of that Company; and needing additional data for the chapter on "Economics of Shopwork" he persuaded that gentleman to promise to furnish some. When the notes came to hand, it appeared that Mr. Canady had confined his remarks entirely to the *economics of design* as practiced in the Company's drafting office. As the said notes are of great value, and as they supplement and, in many ways, endorse the statements herein which precede, the author decided at once to include them in this place; and they are, consequently, given practically verbatim.

PRINCIPLES OF ECONOMICS IN BRIDGE DESIGN FOR SHOP CONSIDERATIONS

"In the following notes relative to the Economics in the Design of Bridges for Shop Considerations, the Drawing Room attached to the Fabricating Shop has been considered as a component and closely related part of the said shop; and points that facilitate the execution of shop drawings have been included.

"In the preparations of a bridge design for a given location, making spans duplicate instead of different lengths has not, in the writer's opinion, been given quite the attention it deserves. Such duplication always means decreased cost of drawings and shop work, and may mean also the adoption of a special fabrication programme for that contract, which would further reduce costs.

"For the same reason the design should lend itself to the maximum amount of duplication in the details. To this end small differences in the cross-section of main members should be shunned. The avoidance of light and heavy trusses, on account of a sidewalk on one side only, is a case in point. This, however, is more applicable to light work and to where the sidewalk load is comparatively small. Otherwise, where there is a material saving in weight by using light and heavy trusses, the difference can generally be accomplished by increasing the thickness of component parts of the various sections without changing the dimensions of the sections themselves, thus preserving the duplication of spacing and the details for the two trusses, as well as those for the lateral-system connections.

"Under modern mill and shop conditions it is economical to build bridges with longer panels than formerly used. Longer panels mean fewer joints and fewer separate parts, with consequent decrease in shop and drawing-room costs.

"If members can be made symmetrical about a point midway between the ends, it counts for economy in the preparation of drawings and templets.

"Often spans have been designed with a small skew where a nominal increase in length, due to making the span square, would have meant a decided saving in the cost of the structure as a whole.

"In the design of a deck-girder bridge, if the span is square, it is economical to arrange the lateral system with an even number of panels, so that the girder can be made symmetrical about the center.

"If the span is skewed, use an odd number of panels, so that the girders can be made alike and their position in the span reversed. Stringers without cover plates should have an odd number of panels of laterals. They will then have the same punching in the top flanges and be made alike for turning end for end, instead of different, as they would be if an even number of panels were used.

"Some Consulting Engineers and occasionally even the designing offices of the Bridge Companies are prone to make their designs so complete as to be almost shop drawings. This does not make for economy, either in

the preparation of the design or in the work of the drawing room. Even where considerable freedom is allowed in the way of modifications to suit the standard methods, details, and equipment of the particular shop doing the work, the fact that such modifications must be arranged for with the Engineer, as well as their actual accomplishment, involves a certain delay and slowing down in getting the work under way in the drawing room. The engineers thereof must necessarily make complete investigation of the geometry, fits, etc., including the requisite number of large-scale layouts, independent of the completeness of the design. The manufacturer is always (and properly so) held responsible for the correct fit of the steelwork. The Engineer's preference for certain types of details, or details that are required by the conditions of erection, should, of course, be indicated as a part of the design.

"The spacing and arrangement of rivets should not be fixed; for the limitations of the specifications as to the maximum and minimum spacing ought to be sufficient. The designer should always keep in mind that modern bridge shops are equipped with multiple punches and various spacing devices, and that contract prices are based upon the largest possible use of such machines. Erection difficulties and impossibilities are often incorporated in such complete designs, which objectionable features must be 'ironed out' by the engineer responsible for the shop drawings before the detailing can go ahead. He has as his particular field the following duties:

"1. To make details for carrying out the specifications and properly developing the strength of the parts connected.

"2. To detail so that the shop can fabricate most economically.

"3. To detail so that the erection methods and the equipment determined upon for the particular bridge shall be not only possible but economical as well. The sequence in the placing of the different members must be taken account of throughout the entire detailing.

"The engineer at the plant, versed in the preparation of shop drawings, is both by experience and environment the best qualified man to meet these three necessities. In fact, the last two are never possible of final solution until after the contract has been signed and the work is being developed in the shop drawing room.

"It is, of course, conceded that, for unusual or monumental structures, the makeup of the details is so interwoven with the general design that the development of the two must proceed together. Even in such cases it is nearly always necessary for the best results that modifications be made as the work progresses in the drawing room.

"The correctness of the foregoing statements has been proved by the actual experience of our Shop Engineers covering their work reaching over a period of the past twenty years. We are now so well fortified with examples

of the best practice for different types of bridges that there is no great difficulty in realizing the true intent of the design in the study of the structure that must be made in the shop drawing room. It would seem to be wise economy on the part of the designing engineer to take full advantage of these facts.

"For full-punched work,* with splices in chords reamed to fit, it is important that the design should provide for the next larger sized rivet in the reamed splices. This saves in the shop one handling of all the main parts, because one size of punched holes will answer for both reamed and unreamed holes. If, for instance, $\frac{7}{8}''$ rivets are being generally used, and allowance is not made in the design for larger punching than $\frac{15}{16}''$, it is necessary to punch $\frac{15}{16}''$ holes in the body of the member and rehandle all the long main parts in order to punch a smaller-sized hole (say $\frac{13}{16}''$ diameter) at the ends where the splice is to be reamed for fit. If allowance is made in the design for $1\frac{1}{16}''$ holes, the punching throughout will be $\frac{15}{16}''$ and the splices will then be reamed out to $1\frac{1}{16}''$, thus saving the extra handling at the punch.

"In heavy, reamed girder-work where several cover plates, side plates, and heavy flange-angles are used, the size of the sub-punched holes should be not less than $\frac{13}{16}''$ diameter. It is difficult and expensive to fit up such work where, because of the size of sub-punched holes, smaller fitting-up bolts are necessary. This is because of the great difficulty in properly pulling together such heavy parts and so many of them with $\frac{5}{8}''$ fitting-up bolts. For work not properly brought together in fitting, the riveting is expensive and apt to be imperfect.

"It almost goes without saying that Forge and Machine Shop work should be kept at a minimum. Bending of long pieces is particularly undesirable, because, when a long angle, for instance, is to be bent, the operation of heating and bending disorganizes and interferes with all the adjacent operations in the shop. Making a bend at each end of a long angle, channel, or beam is not only quite expensive, but next to impossible to do, because of the extreme difficulty in maintaining the correct measurement between bends.

"Curving ends of girders is a considerable item of extra shop expense. It adds to the cost of drawings, templets, laying out, punching, assembling, and riveting, in addition to the extra cost of bending the end angles. In fact, it is doubtful if the æsthetic value is enhanced at all in proportion to the increased price which the buyer must pay. However, where it is decided to use 'round ends,' the exact radius of the curve should be left to the shop detailer, so that standard bending forms may be used.

"Staggered riveting invites shop errors and slows down the work. Preference should be given to rivets placed opposite or in single rows, where the necessities of design do not require that they be staggered. The different members of a span should be designed, as far as possible, to allow

* The author is opposed, on general principles, to punching any rivet holes full size.

the use of power riveters. This is especially applicable to box sections with the flanges of channels turned in. In general, the clear distance in such cases should be not less than $5\frac{1}{2}''$ or $6''$.

"Avoid sections calling for the use of a variety of sizes of shop and field rivets in the same span or structure.

"For narrow, 'I'-shaped sections, preference should be given to four angles with a web plate instead of four angles laced. Often for such narrow sections the latticed type accomplishes very little or no saving in weight, and the shop expense is greater and the result is less desirable from a maintenance standpoint, as compared with the plate-and-four-angle section.

"Quite often a small increase in the thickness of stringer webs will eliminate the necessity for the use of stiffeners.

"Designers very often do not make enough allowance between gross and net section for the proper maintenance of the latter and at the same time for the preservation of rational details at the critical points. For instance, according to the 1920 specifications of the American Railway Engineering Association, if only one $\frac{7}{8}''$ rivet hole is allowed out of an angle, the stagger must be four inches; and other specifications have been more stringent. If the piece happens to be a $6'' \times 4''$ angle with two rows of rivets in the six-inch leg, the detailer is in trouble at once with the location of rivets in the four-inch leg adjacent to the critical section. He is often compelled to give it up and encroach on net section after he has wasted a lot of valuable time in trying to meet the conditions. Any angle with punching in both legs, used in tension, should have two rivet holes deducted from the gross section.

"In plate-girder work, fillers under stiffener angles should not be required to fit tight against flanges. The overrun in width of flange angles, often varying for the different angles on a single girder, means the re-cutting and fitting of fillers to suit. This results in a slowing down and an increased cost in the work of fitting up the girder for riveting. A clearance of at least $\frac{1}{8}''$ should be permitted at each end of filler.

"All unnecessary bevel cuts on the ends of long angles, plates, or other shapes should be dispensed with. These are much more expensive to make on long pieces than on small detail parts. It seems somewhat absurd carefully to cut the end of an angle to bevel, for the sake of appearance, when it is not exposed to view in the finished structure. Indeed, the æsthetic value of such bevel cuts is very much in question, except where a projecting corner is exposed to the skyline.

"The ends of columns for viaducts or other structures resting on masonry can often be made less expensive in both weight and workmanship by using thicker base plates and omitting vertical stiffening angles with their extensive arrangement of wing-plates that are intended to help in the distribution of the load.

"Finally, and in general, the designer should give the most careful atten-

tion to the coördination of the different parts of his design, so that it can be detailed in the simplest and most natural manner with the elimination, as far as possible, of cumbersome or complicated joints and connections. In determining the makeup of the different sections, the detailing possibilities should be given much weight, for quite frequently they are the determining factor."

CHAPTER XXIV

ECONOMICS IN DESIGN FOR ERECTION CONSIDERATIONS

THE tyro in bridge designing, no matter how profound may be his theoretical knowledge, often falls down very hard until he has learned through sad experience that the erectors of structural steelwork have certain standard requirements which *must* be observed. If they be ignored, the work is often seriously delayed; and, as delays in the field are exceedingly expensive, the matter of making the designs so that all the metal will go together easily and expeditiously involves engineering economics of great importance.

The failure to furnish sufficient clearance is generally the tyro's first offense—and it certainly is a serious one. If the designer who commits this blunder could hear what the bridge erectors say about him, his ears would certainly tingle.

Mr. Wolfel's general instructions concerning designing to meet field requirements are as follows:

(1) Allow ample clearance for all entering connections.

(2) Provide erection shelves for girders and beams, particularly when they frame opposite each other.

(3) Have all riveting arranged in such a way that it can follow the erection of the work. Riveting should never be allowed to interfere with the raising and placing of the steel.

(4) Be sure that cross frames in deck bridges can be swung in place without spreading the girders.

(5) Be careful in through plate-girders to arrange the stiffeners so that the floor-beams and stringers can be put in place with the girders in their final position.

(6) Arrange the riveting around the ends of the spans so that the rivets can be driven with the steel in the final position. It should never be necessary to jack up spans to drive rivets.

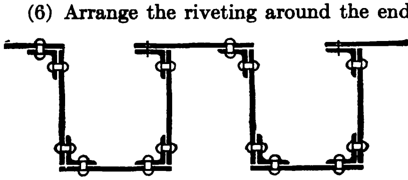


FIG. 24*a*. Trough-Floor Construction for Easy Field Riveting.

(7) In trough floorwork, especially where the under clearance is small, arrange the design so that all field rivets can be driven from the top, as in Fig. 24*a*.

(8) While it has been customary to call for 25 per cent excess for all field rivets, 10 per cent excess should be sufficient, if the rivets are driven with air hammers.

On this subject Mr. Reichmann has written thus:

The designer should always remember to allow plenty of clearance at the ends of sheared members so as to take up variations of shopwork. One-half inch clearance is considered a minimum for sheared ends. For entering connections, plenty of clearance

should be allowed, as a great many of the difficulties in erection are due to lack of clearance; besides, giving reasonable clearance will permit of more rapid shopwork and the avoidance of many errors in the field due to inaccuracies thereof.

It is often advisable to provide, in addition to the usual allowance for expansion, a small amount for erecting the metal, due to what is called the "growth of steel." For viaducts such adjustment should be provided about every 400 feet; and similarly for mill-building work it is necessary to consider effects of adjustment, even if it is decided not to take care of expansion, while in other structures the joints for expansion will also serve the purpose of adjustment.

The expansion points for stringers or elevated railroad-girders, where pockets are used, sometimes have not enough space behind the end stiffeners of the expansion girders to allow for the insertion of rivets through the end connections of the fixed girders. It should be remembered that the expansion stringers and the fixed-end stringers are erected before the rivets in the connection of the fixed stringers are driven.

When columns are set to stone bolts, which have been imbedded in masonry, the holes should be $\frac{3}{4}''$ or $1''$ larger than the diameter of the bolt, so as to provide adjustment to take care of the inaccuracies in setting the bolts in the concrete.

A common mistake in design is to proportion the members with too small a width, causing considerable trouble in packing the pins and in making room for the verticals, pin plates, etc. Another bad feature of narrow chords is that practically all rivets around the connections must be countersunk because of close space, and the ends of the posts must be cut away for clearance, thereby weakening the said ends. By adopting chords of larger widths much better details can be used around the pins at panel-points.

When two or more truss spans are identical, or when they are similar and have the same field connections, the field holes should be reamed to an iron templet, in place of reaming them while the members are assembled. This will facilitate the delivery of the work, and will make identical members throughout the structure interchangeable. The advantages in the field are evident, less time being spent in sorting and finding material.

In the designing of details extreme care should be exercised in arranging all joints and connections, so that the work cannot only be built at the shop for the least cost in labor and material, but also that it may be erected most economically and with a minimum of risk. In the case of bridgework, all connections should be so detailed that spans can be connected and made self-sustaining and safe in the shortest possible time.

Unless for special reasons, it is usually customary to begin the erection of pin-connected spans at the center panel, as this panel has adjustable members and the trusses can be squared up there before proceeding. The details should, therefore, be so arranged that the center panel can be completed and made self-sustaining before the traveler is moved to the next panel. It is the usual custom for the erection to proceed from the center panel toward the fixed end, and after this half of the span is erected, to proceed toward the roller end. Top chord sections in any particular panel are put in place after the posts and bars are erected; and it is especially desirable in heavy work that the details be so arranged that these chord sections can be lifted above the posts and set directly in place without being moved on end or sideways. Therefore, plates connecting two adjoining chord sections in heavy work should always be shipped loose.

Wherever possible, in all truss spans the floor connections should be so arranged that the floor system can be put in place either before or after the trusses have been erected in their final position. It is usually customary, where local conditions will permit, to put the floor system in place first and erect the trusses afterward. This method of procedure has a great many advantages over that of raising the trusses first, viz.: there is a large saving in falsework, as longer panels can be used, putting bents

directly under the panel-points and using the new floor system for carrying traffic and for running out material for the trusses; it permits the posts to be bolted to the floor-beams and released from the tackles on the travelers; it fixes the exact position of the shoes on the piers so that we can proceed with the erection from the center either toward the fixed or the roller end, as we may prefer; it has the advantage of giving more opportunity for jacking up the spans in order to secure proper camber; and it requiers a minimum amount of blocking. There are other features which render it desirable, where possible, to erect the floor system in advance of the trusses. Over dangerous streams, however, where there is a possibility of loss during the erection, it is sometimes desirable to erect the trusses first, so as to have as little material on the falsework as practicable and thus minimize the amount endangered. There are also sometimes certain local conditions which make it imperative that the trusses be erected first; and, therefore, it is important, wherever possible, that details be so arranged that either method can be used. In the erection of through, riveted, lattice spans, it is customary to place the floor system first, then to put the lower chords in position, set up the web members, and put the top chords on last. Therefore, it is more advantageous to have the gusset-plates connecting the web members with top chord riveted to the top chord sections rather than to posts or diagonals, as the rivets in gusset-plates connecting top chords with web members are more easily driven in the web members than in the top chord sections.

In the case of through plate-girders, the details of the floor system should be so arranged that the stringers and floor-beams can be put in place, panel by panel, without the necessity of spreading the main girders. Particularly is this the case in "Rolling Lift Bridges," which, in the majority of cases, have to be erected in an upright position, and where it is extremely dangerous and practically an impossible operation to spread the trusses in order to put in place the floor system.

Top chord sections with half pin-holes, having a hinge-plate on each section, are undesirable. When half pin-holes are used, if possible put a hinge-plate on one section only and make it long enough to rivet or bolt to the adjoining section when in place. Hinge-plates should be arranged so as to give two full pin-holes in center chord sections, and should be put on the ends farthest from the center on the other sections, except in special cases when it is necessary to commence raising spans from the end instead of the center.

Entering connections are usually the most difficult and expensive to make; and where at all possible, entering connections of any character should be avoided, but where they must be used, particular attention should be given to insure necessary clearances. An entering connection is not only an expensive and dangerous operation, but in a great many cases it cannot be accomplished on account of the interference with back walls, adjoining spans, etc.

It is of the greatest importance to allow ample clearance where members are packed inside of chords, posts, etc., as lack of proper clearance causes much trouble and expense, not only augmenting the cost of erection by increasing the time required for making the span safe, but adding materially to the risk. In putting in tie-bars and diagonals, it is customary to connect them on the bottom chord pins first, and then swing them into the chords and posts around the lower pins as a center. All rivet heads coming in the path of bars swung in this way should be cleared. Too much attention cannot be given to this matter of proper clearance. Particularly is this the case in through and deck riveted lattice spans, which are being erected now more than ever before with the use of a derrick car with one boom; and the appliances for pulling tight-fitting members into place are not always present, as was the case formerly when these spans were erected by a gantry traveler. For adjustable rods packed close together, the sleeve nuts should be staggered. Attention should be given to the field connections so that enough space is allowed around all field rivets to enable them to be driven.

All lateral bracing, hitch plates, rivet heads in laterals, etc., should be kept enough below the level of the top chords of girders, stringers, etc., so that the ties when in place will not foul them, it being an expensive operation to notch ties to clear such obstructions.

Where laterals and hitch plates do not interfere with the loading of girders, and are not of such character as will allow them to be easily broken off, they should be riveted to the said girders; otherwise they should be shipped loose or riveted to the braces.

Particular attention should be given to the question of field riveting. Details should be closely examined with a view of minimizing the number of field rivets.

It is not advisable to put two shoes on one bed-plate; but if this cannot be avoided, the bed-plate should be made longer and the anchor holes should be slotted to allow for variations in masonry.

The following are the most important points to be observed in detailing, in order to facilitate and cheapen erection.

(*a*) Avoid as far as possible entering connections.
(*b*) See that proper clearances are given.
(*c*) Minimize the number of field rivets.

Viaducts

When the track is on a grade and the grade is not very steep, make the two bents of one tower alike by adding filler plates on the tops of the up-grade columns. For steep grades make the two bents of one tower the same length, but square the longitudinal bracing. A good detailer will then make the punching the same on all four columns of the tower, but the gusset plates which connect to the longitudinal bracing will be different for the two bents.

When designing columns for the towers, the splices should be indicated so that the column sections should, preferably, be under 40′ long, but should not under any circumstances exceed 60′.

Where two deck girder spans are adjacent, the end cross-frames are placed close to the ends of the girders. The cross-frames cannot be set by swinging in from the end of the span. They must be erected by swinging in from the center of the span. The stiffener angles carrying the cross girders should be built with the back of the angles toward the center of the span.

CHAPTER XXV

ECONOMICS OF REINFORCED-CONCRETE BRIDGES

REINFORCED-CONCRETE structures being the most modern of all the general types of bridge construction, it is not to be expected that the economics of their designing should be so highly developed as in the case of any of the older types. Nevertheless much has been learned about the subject through the numerous investigations made for the author's firms by a number of young computers during the last twelve or fifteen years. Practically all that was known concerning it in 1915 was stated in Chapter LIII of "Bridge Engineering"; and an elaboration of that treatment will be given herein, supplemented by the results of a series of investigations made specially for this book.

The first topic for economic discussion is that of

REINFORCING STEEL

At the present time one of the mooted points in the designing specifications for reinforced-concrete bridges is the proper intensity of working stress for the reinforcing bars. It is generally conceded that it should not exceed one-half of the elastic limit of the metal; and, in consequence, engineering practice in the past has limited it to 16,000 lbs. per square inch, but a number of manufacturers now desire to raise it to 18,000 lbs. per square inch by using a higher-carbon steel—notably re-rolled steel. Of course, the higher intensity saves in the quantity of steel, but generally increases the amount of concrete required; because the higher stress in the steel reduces the moment of resistance of the concrete about six per cent. If the amount of concrete is increased, the net saving is about two (2) per cent in slabs and three (3) per cent in beams; but if the section of the concrete is determined by shear or other considerations, so that no increase is necessary, those percentages will be increased by unity.

There are two grave objections to using the higher steel, viz.:

First. When bent cold it is liable to crack on account of its increased hardness, and

Second. It tends to open up cracks in the concrete.

For these reasons the author is not willing to use such steel in his practice; and, on general principles, he is opposed to the employment of any re-rolled metal, because of the temptation for the manufacturer thereof to run in all kinds of old materials—even wornout Bessemer rails.

Intensity of Working Stress for Concrete

For many years the author's practice has been to stress the concrete in compression only six hundred (600) pounds per square inch; but lately the Joint Committee of the Technical Societies has reported in favor of adopting six hundred and fifty (650) pounds. When a good aggregate is procurable, the author has no objection to this increase of eight per cent; but otherwise he prefers to adhere to his old custom, especially as by so doing he adds only two per cent to the cost of the concrete in those cases where the section can be reduced by using the higher intensity. Very often, though, no such reduction is practicable, and the saving on the entire job reduces to only one per cent.

The actual reduction in the amount of concrete in a beam due to this difference of intensity of working stress is about six per cent, but this is partially offset by an increase in the amount of steel required. The combination of the intensities of 600 for concrete and 16,000 for steel requires a percentage of 0.68 for reinforcing steel, while with 650 and 16,000 that figure is increased to 0.77. Moreover, construction work is simplified by using more concrete and less steel, also better concrete is secured; hence the author does not think that there is much real advantage in adopting the higher intensity. The general complaint from contractors on his work has been "too much steel"; and adopting the higher intensity for concrete would make matters worse. Moreover, the use of heavier concrete sections tends to keep down the shear intensity, which is always desirable.

Frequently there is a choice between a heavy concrete section with no shear reinforcement, and a lighter section with such reinforcement. In superstructure it is generally preferable to adopt the latter and in substructure the former.

Pavings

The subject of the economics of pavings is treated fully in Chapter XXI. The economy lies in the cost of the pavement itself, excepting that brick is heavier than the other paving materials; and, consequently, it adds to the cost of a concrete structure, the amount being from two to three per cent.

It is poor economy to neglect the expansion joints in block pavements; because expansion is sometimes quite destructive, not only causing waves in the pavement but sometimes actually pushing off the curbs.

Handrails

Handrails for reinforced-concrete bridges should always be made of concrete, in order to harmonize with the rest of the structure. A steel railing on a concrete bridge is ruinous to the general appearance, and, therefore, should not be tolerated. The first costs of the two types of railing are about alike; but the concrete one, being heavier, requires more

material for its support. Artistic rails are more expensive than plain ones; hence the selection of the design will often involve a compromise between economics and æsthetics, and will depend greatly upon the amount of money available for the work.

Rails can be built entirely in place, or they may be either partly or wholly cast in sections and set in position. The jointing will require careful attention; for, otherwise, the railing will have a bad appearance and even may go to pieces. A better character of construction can be secured with separately-moulded work; but unsightly joints would offset this advantage. Separately-moulded rails generally interfere much less than other types of rails with the progress of construction; besides, the moulding of the parts can be carried on at odd times in the intervals of other work. Both of these features are money savers.

Designs

The economics of design are rather difficult to determine, as the quantities involved are influenced quite largely by the individual tastes of the designer. The problem is also complicated by the facts that the unit costs of the various portions of a structure may be more or less different, and that the unit costs of different types of construction may be decidedly unlike. In general, it may be said that the unit costs are lower for those structures which have the simplest form-work; and a reduction will also be effected by decreasing the area of form-surface per cubic yard of concrete. For instance, in the case of a wall or slab, the form-cost per cubic yard will vary practically inversely as the thickness of the said wall or slab. Evidently, therefore, it is desirable to concentrate the concrete into a few large members, rather than to employ a great number of small ones.

It should be noted that reinforcing bars less than $\frac{3}{4}''$ in diameter command higher pound prices than do the larger bars. The extras for these small bars may be found in *Engineering News-Record* in the first issue of each month.

The economics of the designing of the different parts of reinforced-concrete structures will now be discussed in logical order.

Slabs

A primary economic problem in slab designing is that of two-way *versus* one-way reinforcing. Two-way reinforcing involves less concrete but more steel than does one-way reinforcing; hence it has but little advantage, unless the reduction of dead-load to a minimum be of prime importance.

Different arrangements of slab steel are discussed on pages 918 to 921, inclusive, of "Bridge Engineering"; and it will be noted therefrom that there is very little difference in the weights of the various types. Lighter

arrangements of steel are generally used in buildings; but roadway slabs, in view of the heavy concentrated-live-loads that they have to carry, and which produce large shears throughout and positive moments over almost the entire span, require carefully and liberally designed reinforcement.

Barring most of those in railway bridges, slabs are usually continuous over panel points, excepting at the expansion joints. There is but little difference in the actual costs of continuous and non-continuous slabs; but continuity is desirable from the standpoints of paving and drainage; also with continuous slabs T-beam construction can be employed—involving the saving of much material in girders. The continuity of slabs and girders complicates construction problems—sometimes very seriously. The various processes of the construction of a proposed design should be studied through completely in order to make certain that no impracticable or unnecessarily expensive work is involved. Frequently important savings can thus be effected; and sometimes it is found cheaper in the long run to use more concrete than the minimum practicable amount. For instance, a sidewalk slab usually rests upon the curb, which in turn rests on the roadway slab; and in the designing it is advisable to provide for a construction joint at the top of the roadway slab, because it would be both difficult and expensive to pour the curb and the sidewalk slab simultaneously with the roadway slab. Such construction joints can be arranged for by the designer without involving much extra expense, provided that he gives the matter proper consideration at the outset.

Girders

Girders are of two main types, single or continuous; and there is no great difference in their costs, there being more concrete but less steel in the simple-span type. The two-span-continuous type is nearly always a little more expensive than the simple-span type. The simple-span girder can be moulded separately and set in place. This is not a paramount feature in highway bridges, but it is often all-important in railway structures.

Comparing simple girders and continuous ones of three or more spans, the following general observations may be made:

If there is no T-beam action, the simple spans will be the more expensive; because the section will be determined by the moment at mid-span in the simple girder, and this is greater than any of the moments of the continuous girders. Again, higher unit stresses than ordinary are allowed over the supports of continuous girders. For the T-section, if the bottoms are straight, the continuous type will be the more expensive, having more concrete and more steel than the simple type. But if the bottoms of all girders are curved, the continuous girders will be the cheaper, there being decidedly less concrete required for them. When the bottoms are straight for simple-span girders and are curved for continuous girders, the curves on page 1323 of "Bridge Engineering" indicate the relationships. It will be

noted therefrom that for light loads the simple spans have less concrete, and that for heavy girders there is but little difference in the concrete quantities, excepting that for exceedingly-heavy, long-span girders the continuous type is the lighter. There is always more steel required for the continuous type than for the non-continuous one.

From the foregoing remarks it is evident that simple-span girders are generally cheaper than the continuous ones. It will be found, however, that the bents or other supports are cheaper for continuous girders than for simple ones, and that floor joints in simple spans are expensive, also that the continuous girders give a solid, monolithic structure. The continuous-girder construction is very generally used in highway bridges; but the railway companies have adopted as standard the simple-girder type. However, nearly all railway concrete-bridges up to the present have been of solid-slab construction, rather than of that of the slab-girder.

The foregoing comparisons are based upon girders in which the reinforcement was liberally proportioned for positive moments, negative moments, and shears, making full provision for impact and a small allowance for uncertainties of stress distribution in continuous girders. It is possible to skimp the reinforcing of concrete girders considerably, and this practice in highway bridgework is altogether too common. It is an evil that should be stamped out, if not by the engineering profession, then by the laws of the land; for while it is most reprehensible to skin a steel bridge in which the skinning cannot be hidden from the expert eye, it is criminal to trim down to dangerous limits of strength a reinforced-concrete structure in which the flaws and weaknesses are buried out of sight.

The character of the foundations should be duly considered in deciding between simple and continuous girders; for, if there is danger of settlement, the simple-girder type is far preferable—in fact, it is obligatory.

The balanced-cantilever type of girder is beginning to be used, each monolithic unit consisting of a pier and two half-spans. In this the foundation pressure is centric for dead load and for live load over the two arms; but with the latter loading on one arm only, the pressure on the base is decidedly eccentric, subjecting the pier shaft to bending. This type of layout permits of a very shallow depth at the center of the span, and is thus specially applicable to long spans, where the weight of the concrete in the central portion of either simple or continuous girders is an important factor. The balanced-cantilever girder usually shows for the superstructure a small economy over either the simple or the continuous girder for short spans and a larger saving for long ones. The substructure, though, is always more expensive, as it has to be designed for the unbalanced load on one arm. If the live load is small in comparison with the dead load, this increase in sub-structure cost is not great; but otherwise it is so large as to outweigh the economy in the girders themselves. This type of layout should not be used on soft foundations

or with pile bearings when the live load is comparatively great, on account of the tendency to rock the piers. Where the ends of the arms come together it is necessary to insert a detail that will take up shear but not moment; for, otherwise, there would be a sudden break in the grade that might give serious trouble. In the author's opinion this type is never suitable for carrying steam-railway loadings and is none too good for electric-railway structures, although satisfactory enough for highway bridges. In order to determine for any case the relative economy of cantilever and ordinary types, estimates for both superstructure and substructure will have to be made.

Columns

Columns are generally square or rectangular in cross-section for constructive and æsthetic reasons. A round or octagonal column is really a better structural member; and, if the lines of the bridge are worked out in accordance with it, there should seldom be any difficulty about the matter of appearance. A round column can be hooped or banded better than any other type. Frequently, for the sake of appearance, the size of a column must be made greater than that necessitated by theoretical requirements.

Footings

Footings may be either plain or reinforced; and the question as to which style to adopt is one solely of economics, because, as they are buried out of sight, the consideration of æsthetics will not apply. If the area of the footing is but little larger than that of the column supported, plain concrete will be the cheaper; while for a spread foundation the reinforced type will nearly always be found more economical. If a footing has to be poured under water, plain concrete should invariably be employed; and in wet-excavation work in general it is preferable, because it is often difficult to prepare the bottom of the pit properly, and to stop absolutely the flow of water from below. Such a flow is liable to wash out the cement from the lower part of the footing; and thus it would destroy most of the value of the reinforcing.

Plain footings are made of 1 : 3 : 5 concrete or sometimes 1 : 3 : 6; but the latter, in the author's opinion, is too weak. The use of 1 : 2 : 4 concrete permits thinner footings, but this is not of much importance when plain-concrete bases are used.

Highway Girder Bridges

In respect to the economics of girder bridges resting on columns, the following points must be considered:

First. The panel length, when cross-girders are employed.

Second. The number and spacing of the longitudinal girders.

Third. The number of columns per bent.

Fourth. The span length.

Fifth. The use of reinforced concrete piles to carry the footings.

The panel length adopted is usually not of great importance from the standpoint of economy. Lengths of from eight to ten feet are generally employed; but a considerable variation from these values will cause little change in the combined cost of the slabs and cross-girders. A reduction in concrete quantities can frequently be effected by using long panels, and by carrying the slabs on short stringers supported by the floor-beams; but the extra form work required will generally overbalance this saving in volume.

The number and spacing of the longitudinal girders will depend upon the width and the height of the structure, the span-length, and the load to be carried. For a high structure in which the economic span-length is fairly great, it will nearly always be found best to employ two lines of girders, the spacing thereof being equal to about five-eighths of the total width of the structure; but for bridges much over sixty (60) feet wide the use of three or even four lines may be preferable. The slab in such structures is carried on cross-girders and cantilever-beams. For a low bridge in which the economic span length is short, it will generally be the cheapest to omit the cross-girders, except at the bents, and to employ several lines of longitudinal girders. The wider the structure, the more likely will this arrangement prove to be economical; and very heavy loads also favor its adoption. For a structure in which the span-length is from one-half to two-thirds of the width, it will usually make little difference which of the two types is adopted, unless the height is rather large; and even in extreme cases the variation between the two is not likely to exceed ten per cent. Ordinarily, it will be found more desirable to use two lines of girders, with cross-girders and cantilevers about eight or ten feet centers.

The proper number of columns per bent depends on the number of longitudinal girders. When there are only two lines, two columns will, of course, be employed. When there are several lines of girders, there should generally be one column per girder in low structures, and two columns per bent in higher ones. In this latter case a heavy cross-girder will be required at each bent to carry the longitudinal girders.

The economic span-length is affected by the height and the load, being larger for greater heights and smaller for heavier loads. An approximate value thereof is given by the formula,

$$\boxed{l = h\left(0.3 + \frac{2{,}000}{w + 1{,}000}\right),} \qquad \text{[Eq. 1]}$$

in which l = economic span length in feet, measured from center to center of supports,

w = load in pounds per lineal foot of girder (excluding its own weight),

and h = fixed height of structure in feet.

The quantity h represents in any given case the height which is fixed, such as the height from grade to top of footing, height from grade to bottom of footing, height from underside of girder to top of footing, or height from underside of girder to bottom of footing, as the case may be. There is always a considerable range of lengths for which the quantities remain nearly constant. The formula gives values a trifle greater than those for which the quantities are a minimum, since the use of heavier sections will reduce slightly the unit costs of the concrete.

Reinforced-concrete piles should be used under footings when a suitable foundation is to be found only at a considerable depth, or when a very large footing-area would be required in order to reduce the pressures to a proper amount. A comparison must be made for each case as it arises, allowing properly for the costs of the column shaft, the footing, the piles, and the excavation. This latter item must not be overlooked.

The curves of Figs. 56*t* to 56*y*, inclusive, of "Bridge Engineering," will be found of great value in studying the questions of economy of reinforced-concrete-girder bridges, as most of the points involved can be settled directly thereby.

Arch Bridges

Economic Rise with Span-Length Unchanged

The economics of arch bridges are much more complicated than those of girder bridges. The important factors are the costs of the arch ribs and those of the piers or abutments; and the main economic point to determine is that of ratio of rise to span-length. For any fixed span-length, the greater the rise, up to a limit of one-third of the opening, the smaller will be the cost of both arch-ribs and piers. By increasing it further, up to the limit of one-half of the opening, the cost of the rib will be but little augmented, and the cost of the pier above the springing will be increased, while that of the portion thereof below the same will be reduced. If the increase in rise is secured by lowering the springings, the greater the rise the greater the economy of material and cost; but if the increase must be secured by raising the grade, the springing remaining at a fixed elevation, it will rarely be economical to increase the rise above the limit of one-third of the opening. The exact limit in any case will depend upon the distance from the springing to the bottom of the base, and upon the massiveness of the pier-shafts above the springing; also upon the spans of the arch-ribs resting on the pier, and upon the character of the substructure employed. If the springing is but little above the top of the base, a comparatively-low rise will be economic.

If the pier carries two arch-spans of the same length, and if the live loads are small as compared with the dead loads, a low ratio of rise to span-length will be economic. On the other hand, if the distance from the springing to the bottom of the base be great, the live load large as compared with the

dead load, the two adjacent arch spans of different lengths (or, still more important, if there be only one arch), and the substructure work expensive or the foundation of low bearing-value, the ratio of rise to span length should be large.

Frequently an arch abutment retains a fill, in which case a very low rise is likely to be economic, in order that the larger thrust of the rib may oppose the earth-thrust on the abutment. In such an event it may be economic to make the springing higher. This is the only exception to the general rule that, for maximum economy, the springing should be placed as low as clearances, waterway requirements, or due consideration of æsthetics will permit.

Economic Span-Length with Rise Unchanged

In most instances there is little chance to vary either the grade or the elevations of the springings to any great extent; hence the principal economic problem is the determination of the best span-length. The principal factors to be considered are the following:

A. The rise of the arch.
B. The distance from springing to bottom of base.
C. The character of the substructure work.
D. The massiveness or lightness of the piers, determined from the æsthetic viewpoint.
E. The ratio of live load to dead load.
F. The type of arch-ring—whether solid-barrel or two or more separate ribs.
G. The equality or inequality of lengths of adjacent spans.
H. Arbitrary requirements fixing clearances of ribs or positions of piers.
I. Other special conditions.

The rise of the arch is evidently of paramount importance; because the greater it is the greater will be the economic length of span.

The distance from springing to bottom of base is another very important factor. In general, it may be stated that, for ribbed arches, when the adjoining spans are of equal length and when the springings are but a short distance above the bottom of the base, a ratio of rise to span-length of one-third or even less will be quite economic; while, if the said springings are a considerable distance above the said bottom, a ratio of one-half will be better. Generally speaking, it may be said that low ratios of rise to span are more pleasing to the eye than higher ones, so that the adoption of longer spans is preferable from the æsthetic standpoint. Also longer spans involve larger members, and consequently lower unit costs, so that the economic span-length is somewhat greater than that which gives minimum quantities of materials.

Difficult foundations favor long spans, not only because of the reduction in the number of piers but also because the unit costs for small piers are much higher than those for large ones. On the other hand, if the foundations are very deep, the effect of unbalanced thrusts becomes of great importance; and this favors shorter spans. Poor foundation conditions also militate for shorter spans, as do invariably pile foundations.

If it be decided for the sake of appearance to make the piers heavy and massive, this will tend towards greater span-length; because, in that case, up to a certain limit, an increase of span will augment the size of each individual pier but little, if any. It will rarely pay to reduce the span-length, if such reduction will not decrease the size of the pier or piers.

Light live loads in proportion to the dead loads tend, for economy, towards the adoption of longer spans, especially when the adjoining spans are of the same length. With such light live loads the economic span-length is not greatly affected by the distance from springing to bottom of base. When a pier carries one arch span only, the ratio of live load to dead load is of much smaller importance than it is in the case where there is a succession of spans.

The type of arch ring, whether it be of one solid-barrel or of two or more arch ribs, often affects materially the economic span-length, but to what extent it is difficult to predict in advance of designing and estimating. The piers of the solid-barrel type are generally more expensive than those of the ribbed type, as are also the arches; but in most cases the piers are comparatively the more expensive; and this favors the employment of longer spans. Also, it will frequently be found that increasing the span-length will augment but slightly the quantities in the arch rings of the solid-barrel type—which also favors longer spans. The economic ratio of rise to span-length for solid-barrel arches may generally be taken at 0.25, varying, of course, with the other factors previously discussed. The foregoing conclusion may seem to be in contradiction to the well-known fact that long spans are nearly always of the ribbed type. This, however, is because the solid-barreled arch is generally used for low rises, which necessarily means comparatively short spans, while the open-ribbed arches are employed for nearly all arch structures of high rise. The comparative economics of these two types will be discussed later in this chapter.

When adjoining spans must be unequal, the inequality, for economy's sake, should be made as small as practicable. This is very important if the springings are far above the base; but is of small consequence if they are not. If the springing of the smaller arch can be located well above that of the larger one, it may be possible so to adjust the spans and rises that there shall be very little eccentricity of pressure on the base, in which case the pier will not be much more expensive than it would have been had the adjacent spans been of equal length. The appearance of the piers

will demand special study, if the springings are located at different elevations; otherwise, the principles of æsthetics are likely to be violated.

The influence of earth-thrust on an arch abutment has already been mentioned. Where such thrust exists, it will favor the use of longer spans, in order that the larger arch-thrust may counterbalance the said earth-thrust. In a long bridge of many spans, the abutments will not cut much figure; but if only two or three spans are to be adopted, they may be of more importance than the intermediate piers. If there is no earth-thrust on the abutment, its cost will augment as the span-length increases; but if there is such a thrust, the cost may reduce when the span-length is made greater.

In many layouts the positions and even the sizes of certain piers are fixed, and specified clearances (both horizontal and vertical) must be maintained under certain of the spans. These conditions, of course, must be taken as a basis, and the layout adapted to suit them. Special governing conditions must often be observed, such as the character of the surroundings, the relative importance of æsthetics and cost, or the adoption of some special architectural treatment.

In a discussion of economic span-lengths of arch bridges it is impracticable to give exact figures, for there are too many variables concerned. Whenever there is any choice, it will be necessary to make preliminary designs for at least two span-lengths; and, if one of these proves to be somewhat cheaper than the other, a third length should be figured. There is no other way to ensure that the truly economic length has been selected, unless it happens that the figures can be compared with those for a very similar structure.

Comparison of Solid-Spandrel versus *Open-Spandrel and Solid-Barrel* versus *Ribbed Structures*

Arch-spans can be divided into two general classes, solid-spandrel and open-spandrel. In the first form the arch-rib must be solid, while in the second it may be either solid or ribbed. Considering the two solid-barrel types, the filled-spandrel has no floor-system or cross walls, but instead an earth fill between the spandrel-walls. For very low rises the walls and filling are cheaper than the floor-system and columns; but for high rises the reverse is true. The great weight of the filling makes the piers expensive in the cases of pile foundations and bearings upon comparatively soft soil. The spandrel-filled arch is often used on railroads when it is not the most economic type, in order that the weight and inertia of the filling may absorb the impact of moving trains.

In highway structures, the open-spandrel type is generally preferred to the filled type, for various good and economic reasons, among which may be mentioned the fact that, with the latter type, it will be found desirable, for the sake of appearance, to make the ring the full width of the

deck; whereas for the former type it will be satisfactory to carry a part of the deck on cantilevers. The consequent narrowing of the arch-rings and shortening of the piers involve quite a saving in cost.

Comparing, in open-spandrel structures, the solid-barrel type and the ribbed type, it will be found that the latter is cheaper, except in the case of very low ratios of rise to span-length. In the solid-barrel type there is one wide, rather-thin ring carried on a wide, comparatively-thin pier; while in the ribbed type there are two or more thicker and rather-narrow ribs, carried on piers which must be somewhat wide as seen in side elevation, there usually being a separate shaft for each line of ribs. For arches of considerable rise in which the live load moments are the only ones of importance, the thick, narrow rib is much the cheaper; but, as the rise is reduced, temperature and arch-shortening stresses increase in importance, and it becomes more economical to reduce the thickness and make each rib wider, until eventually the solid-barrel rib is reached. For any special case, comparison can be made by means of the curves on pages 1332 and 1333 of "Bridge Engineering." It must not be forgotten that, with the ribbed type, cross-braces between the ribs are generally necessary. The type of pier required is also important. If a separate shaft can be used for each line of ribs, the ribbed type will usually be the more economical; but in many cases, as in most rivers, solid pier-shafts must be employed in any event. Frequently, in river crossings, the springings are located below the high-water line; and the adoption of solid barrels then becomes almost imperative.

Hingeless and Three-Hinged Arches

Comparing the hingeless and the three-hinged types for reinforced-concrete arch-bridges, it will be found that the latter is cheaper for low rises, and the former for high ones. The principal objection to the three-hinged type is its awkward appearance, due to the fact that it is thicker in the haunch than at the springing line; and since the concrete-arch bridge is often selected from æsthetic considerations, this is an important matter. If the three-hinged rib be thickened at the springing, in order to make its appearance satisfactory, it will rarely, if ever, prove to be cheaper than the hingeless type.

Arch with Steel Bottom Chords

An unusual economic problem arose in the design of the author's Twelfth Street Trafficway Viaduct in Kansas City, Mo. This is a double-deck, reinforced-concrete-girder structure; but there was one portion of it where a 134-foot span was required over some railroad tracks. An arch span was adopted; but the springings were high above the foundations, which were on piles; and the area allowable for the piers was restricted by other tracks. The question of what to do was finally solved by putting in

encased bottom chords of eye-bars at the elevation of the lower deck, so as to take up the arch thrusts. The available clearance was very small; so that the floor-beams of the lower deck had to be made of steel I-beams encased in concrete and riveted to steel hangers, also encased.

REINFORCED-CONCRETE TRESTLES FOR STEAM RAILWAYS

A number of reinforced-concrete trestles have been built of late years for steam-railways. The usual type for river crossings has been solid-concrete slabs on concrete piles. The economic span-lengths for such trestles are very short, usually from ten (10) to fifteen (15) feet; but most of the structures have had spans of fifteen (15) or twenty (20) feet on account of waterway requirements. Solid slabs have also been used extensively for grade-crossing-elimination work in cities, the substructure generally consisting of either solid cross-walls or cross-girders resting on a row of small columns carried on a continuous footing. The economic span-lengths for such structures are a little greater than those for the slab-pile-trestles; but this economic question is of very small importance, since the span-lengths are generally fixed definitely by other considerations.

The slabs for most of these trestles have been separately moulded and afterwards set into place by derrick cars or locomotive cranes. In some cases this was the cheapest possible method of construction; but in others the need for maintenance of traffic demanded it. In many cases also the headroom was limited, and falsework beneath was not permissible.

For the purpose of determining the economics involved, the author has had made in his offices during the last three or four years a large number of estimates for steam-railway, reinforced-concrete trestles of the slab-girder type, for span-lengths varying from twenty (20) to fifty (50) feet, and for heights varying from twenty (20) to sixty (60) feet, measuring from base of rail to bottoms of footings. Both simple and continuous girders have been computed, the former proving somewhat the cheaper. The substructure considered consists of bents composed of two battered columns with a cross-girder at the top, the columns being supported by either one continuous footing or two separate footings. For the higher trestles it proved to be economical to use longitudinal struts between columns in alternate spans, thus making a succession of towers with a single span between each of them. The economic span-length for such trestles varies from one-half to six-tenths of the height.

In the second edition of "Bridge Engineering" (fourth thousand), which will probably be issued in 1922, there will be a lengthy Appendix for the purpose of recording the results of all of the author's studies (excepting the economic ones reproduced in this treatise) on the subject of bridges made since July, 1916, when the first edition of that work appeared. The said Appendix will contain an extensive series of diagrams giving curves of quantities for different types of steam-railway, reinforced-concrete trestles.

Comparing such structures with those composed of steel girders carrying ballasted decks on concrete slabs, there is very little difference in cost, the reinforced-concrete trestles, pure-and-simple, being a trifle cheaper for twenty-foot spans and a little more expensive for fifty-foot spans. But if the steel girders are encased in concrete, the reinforced-concrete trestle will always be found the cheaper.

Retaining Walls

Reinforced-concrete retaining walls show a small saving over plain-concrete ones for even small heights. However, it is better to use plain concrete up to heights of ten (10) feet, measuring from bottom of footing to grade, as the section of a low reinforced-concrete retaining wall is too small, for the reason that a certain amount of massiveness is necessary in such constructions.

In respect to the economics of cantilevered and counterforted walls, the former type is the cheaper up to about twenty or twenty-five-foot heights, above which limit the latter type is the more economic. The discussion of "Footings" previously given in this chapter will apply to the footings of retaining walls.

CHAPTER XXVI

ECONOMICS OF STEEL ARCH-BRIDGES

For some twenty-five or thirty years American bridge engineers have been sadly in need of reliable information concerning certain fundamental economic functions of steel arch-bridges, and especially the relative costs of such structures in comparison with the corresponding truss bridges. The want of such information has resulted in the failure of American engineers to develop the steel arch and to use it in places where it would be both more economic and more æsthetic than the simple-truss bridge. Wherever the governing conditions of crossing are suitable for an arch structure, that type of bridge is to be preferred; for, as we now know, the arch, in all places where its use is really justified, is less expensive than the truss bridge.

For two decades or more, the author had been urging the engineering profession to make certain investigations which would settle for all time every uncertainty concerning the economics of designing steel-arch bridges, as well as the relative costs of these structures in comparison with the corresponding truss bridges. As long ago as 1897, when he wrote "De Pontibus," he appealed to the engineering profession as follows:

> In concluding this chapter, the author desires to call attention to the fact that there is still a great deal to be learned about the designing of arches; and to suggest that some professor of civil engineering, who is well posted on bridge designing and who has time to spare, could spend several months to the great advantage of the engineering profession in determining the proper relations of span-lengths, rise, arch depth, width between exterior arches, etc., for the varous styles of arch, and in ascertaining the relative economies of the latter.

During the eighteen years which elapsed between the issuing of "De Pontibus" and its successor, "Bridge Engineering," the author made repeated attempts to find some bridge engineer who would be willing to undertake the task which he had suggested—but all to no avail; and in the last-mentioned treatise he made another forcible appeal to the profession to undertake the requisite investigations, speaking as follows:

"For many years the author has been endeavoring to establish some approximate relation between the weights of metal per lineal foot for trusses and laterals of arch bridges and those for the corresponding simple truss bridges; but has met with very little success. He once submitted the question to his brother bridge specialists of America, but they were unable to throw any light upon the subject, because their opportunities to

design and build arch bridges had been few and far between, and because the ratio of rise to span has a great effect upon the weight of metal in an arch. Of course, there is for any span length some economic value of that ratio; but it is not yet known,,and it probably varies more or less not only with the span but also with the type of construction. The only practicable method of determining the original question would be to settle first that of the economic ratios of rise to span, design a few arch bridges with the said ratios, and make the comparison. This would have to be done for the solid-rib, the braced-rib, and the spandrel-braced types to make the job complete, adopting for the first set of curves the three-hinged type, and afterward modifying the results for the other three types of hinging. It is evident that the amount of work involved in such an investigation would be immense. It should be done by an experienced bridge designer, as the results would be worthless if obtained by any other investigator. The author suggests that one of his younger brother-specialists undertake the investigation."

The result of this appeal, as in all previous attempts, was absolutely *nil;* and in 1917, despairing of ever being able to induce anyone to assume the obligation, the author himself shouldered the burden by making, with the aid of one of his assistants, all the calculations needed to determine every desired point, and from the results thereof prepared a paper on "The Economics of Steel Arch Bridges," which he presented to the American Society of Civil Engineers. As soon as he learned of its acceptance by the Publication Committee, he sent a circular letter to all the bridge engineers in America whose addresses he could obtain, as well as to a number of prominent engineers abroad, inviting them to discuss the paper, and enclosing the following synopsis:

Up to the present time, nothing at all certain has been known concerning the economics of steel arch-bridges, the weights of metal required to build them, or how they compare in cost with the corresponding steel truss-bridges.

The objects of this paper and its anticipated discussions are to settle finally every important economic question that can arise in the designing of steel arches; to give formulæ and diagrams for determining, with a fair amount of accuracy, the weights of metal in both arch-bridges as a whole and the arches themselves; and to indicate the relations between the weights and costs of arch-bridges in comparison with those of the corresponding truss-bridges.

There are eight economic problems set for solution; and all of them have been solved—the first two by employing certain formulæ given in "Bridge Engineering," and the other six by means of a large number of special arch-designs and the resulting estimates of weights of metal. Incidentally, during the investigation there have arisen and been solved a few minor questions which may properly be termed 'side-issues. The computations have been made for both railway and highway arch-bridges; and the weights of metal are plotted for both carbon-steel and nickel-steel structures. Several diagrams are given to show the percentage effects of weight-increase due to departure from economic conditions.

The need for the special knowledge concerning arch-bridges presented in this paper has been recognized during the last three decades by American structural-steel-engineers; but the authors of books on bridges (this author included) have hitherto

avoided the issue because of the immense amount of work involved in the solution of the various problems. The author has tried of late years on several occasions to persuade some of his brother engineers to undertake the task; but not meeting with any success, he finally decided to do the work himself. The results of his efforts are herewith presented to the members of the Society with a most earnest request for a thorough discussion, particularly by those who specialize in bridgework.

The principal economic problems concerning the design of steel arch-bridges which have been solved for this paper are the following:

First. The economic ratio of rise to span-length.

Second. The economic depths for the ribs.

Third. The economic location for the crown-hinge in three-hinged, spandrel-braced arches.

Fourth. The ratios of weights of metal required for the solid-rib, the braced-rib, and the spandrel-braced types.

Fifth. The ratios of weights of metal required for the hingeless, the two-hinged, and the three-hinged types.

Sixth. The economics involved by making arches three-hinged for the dead load and two-hinged for the live load.

Seventh. The economy of the cantilever-arch with suspended end-spans, as compared with an ordinary arch and two flanking simple-truss-spans.

Eighth. The ratios of weights of metal required for certain portions of arch bridges as compared with the corresponding portions of simple-truss bridges of the same span and same live-load-carrying-capacity.

In answer to this request, a number of engineers, both at home and abroad, discussed the paper; and their discussions have been printed in several copies of the Society's "Proceedings." These discussions have been thoroughly analyzed and replied to in the closure of the memoir.

With one exception, the discussions did not result in the modification of any of the author's findings—and in that one the change was not serious. It was pointed out by him that the first two problems were solved by the use of certain semi-rational, semi-empirical formulæ for weights of metal in arches established in Chapter XXVI of "Bridge Engineering," but that the other six were settled by means of actual computations of weights of metal determined from specially-prepared diagrams of stresses and sectional areas of main members.

Mr. Charles Evan Fowler in his thorough and elaborate discussion, including a tabulation of the salient features and dimensions of one hundred of the world's largest steel-arch-bridges, showed that the author's finding for economic ratios of rise to span-length agreed fairly well with the averages computed from existing structures, but that his deduced economic ratio of depth of arch-ring to span-length did not; whereupon the author agreed to settle this question beyond a doubt by submitting it to the incontrovertible test of making actual designs and estimates of weights for arches of 500-feet span, of economic ratio of rise to span-length, and having regularly-varying depths of arch ribs, so as to determine the depth producing the least weight of metal. This was done; and the results of the special calculations were given in the résumé, which showed that, for short-span railroad arch-bridges, the previous finding was correct, but that,

for the corresponding long-span structures, it was somewhat too great—also that the conclusion to the effect that the economic depth for arch ribs is the same for both railway and highway bridges is wrong. The reason for the variation in the two findings is that the weight formulæ, upon which the first investigation was made, were of too empirical a nature to warrant their use for an exact determination of economic depth of arch rib.

From the original paper, the discussions, and the résumé, the following answers to the set questions have been reached:

First. For three-hinged arches with the grade line approximately tangent to the top chord of the arch at the crown, the average economic ratios of rise to span-length are as follows:

Solid-rib structures	0.2
Braced-rib structures	0.225
Spandrel-braced structures (with hinge above)	0.25

These values may be either increased or decreased by 0.025 without making any material difference in the economics.

For three-hinged, half-through arch-bridges, the average economic ratios for rise to span-length are as follows:

Solid-rib structures	0.225
Braced-rib structures	0.3

For three-hinged, high-deck arch-bridges, they are as follows:

Solid-rib structures	0.25 to 0.28
Braced-rib structures	0.33 to 0.38

For two-hinged arches and combined two-hinged and three-hinged arches, the economic ratios of rise to span will be practically the same as for three-hinged structures.

For the hingeless arch, a somewhat greater ratio of rise to span than that for the three-hinged arch is economical. The single test of this made for the 500-ft. span indicates that the best ratio is about 0.28 with low-grade deck, 0.33 for half-through arches, and 0.38 with high-grade deck.

Second. In respect to solid-rib arches, the question of economic rib-depth does not arise; for the depth should always be made as great as a proper consideration of the section for resisting compression will permit, and with due regard to shipping restrictions concerning limiting sizes of single pieces. For braced-rib, three-hinged arches in steam-railroad bridges, the economic rib-depth varies from 7.8% of the span-length for 100-ft. spans to 5.3% thereof for 1000-ft. spans; and for highway bridges the corresponding variation is from 5.8% to 4.2%, as shown in Fig. 26*a*.

The effects on rib weights from using uneconomic rib-depths in braced-rib arches for both railway and highway bridges are given in Fig. 26*b*.

Third. It was found for all cases that the most economic location for the crown-hinge in a spandrel-braced arch is in the top chord. In braced-

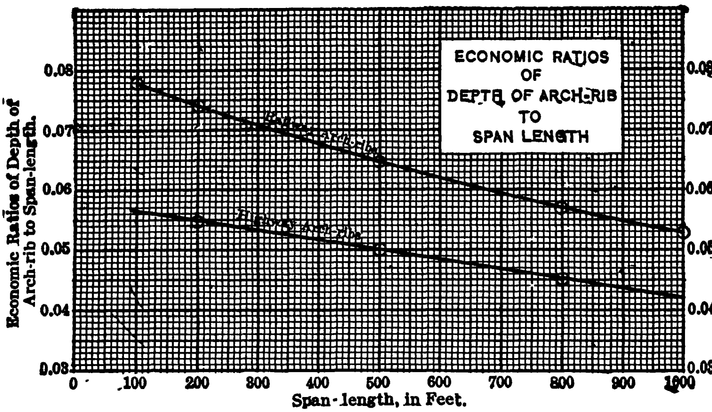


FIG. 26*a*. Economic Ratios of Depth of Arch-Rib to Span Length.

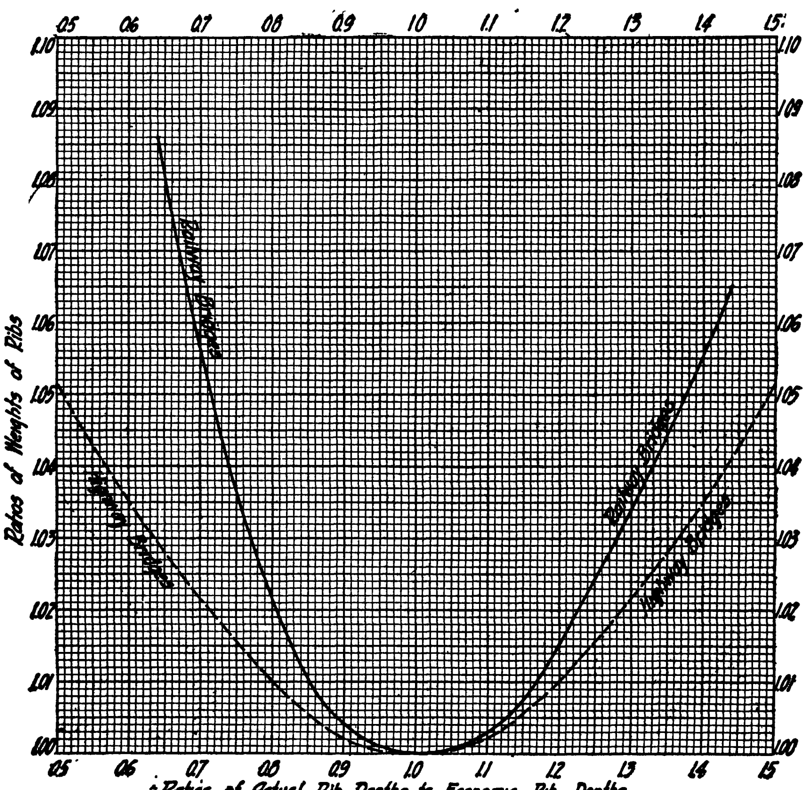


FIG. 26*b*. Effects on Rib-Weights from Using Uneconomic Rib-Depths in Braced-Rib Arches.

rib and solid-rib arches, however, the hinge should always be placed at mid-depth, so as to distribute properly the thrust over the two chords.

Fourth. Comparing the economics of solid-rib, braced-rib, and spandrel-braced arches, it was found that the first mentioned is always considerably heavier than either of the others; and, under normal conditions of the metal market, it is also more expensive. The spandrel-braced-arch requires for long spans a little more metal than the braced-rib arch; but,

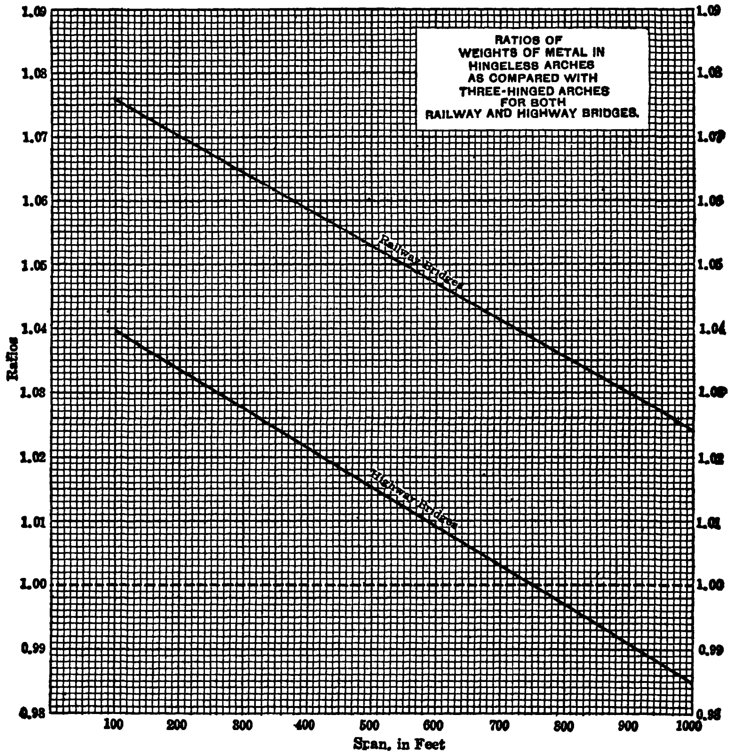


FIG. 26c. Ratio of Weights of Metal in Hingeless Arches as Compared with Three-Hinged Arches for both Railway and Highway Bridges.

in case of cantilevering, this would generally be offset by the extra quantity of erection metal needed for the braced-rib structure. In short spans, the spandrel-braced arch is a trifle the lighter.

Fifth. Comparing three-hinged, two-hinged, and hingeless arehes, it was found that the three-hinged is a little heavier than the two-hinged, but nearly always lighter than the hingeless. The variations in weight among the three types, however, are never great. In Fig. 26c are given ratios of weights of metal in hingeless arches as compared with three-hinged arches for both railway and highway bridges.

Sixth. The combination of three hinges for dead load and two hinges for live load produces very little saving over the three-hinged type; but it adds materially to the rigidity. In the writer's opinion, the combined type is preferable to any of the others.

Seventh. When an arch is flanked by other spans than arches, there is generally quite a little economy involved by cantilevering the ends of the arch and shortening the lengths of the simple spans. The best proportions for lengths of cantilever arm and suspended span to total length of flanking span are, respectively, 0.4 and 0.6.

Eighth. The ratios of weights of arches to the weights of the corresponding simple spans for both railway and highway bridges have been determined and plotted. As was anticipated, the arch usually effects a greater relative economy in highway structures than in railway structures of the same span length; and the longer the span the greater always is the proportionate saving of metal. The results of this investigation are shown in Fig. 26*d*.

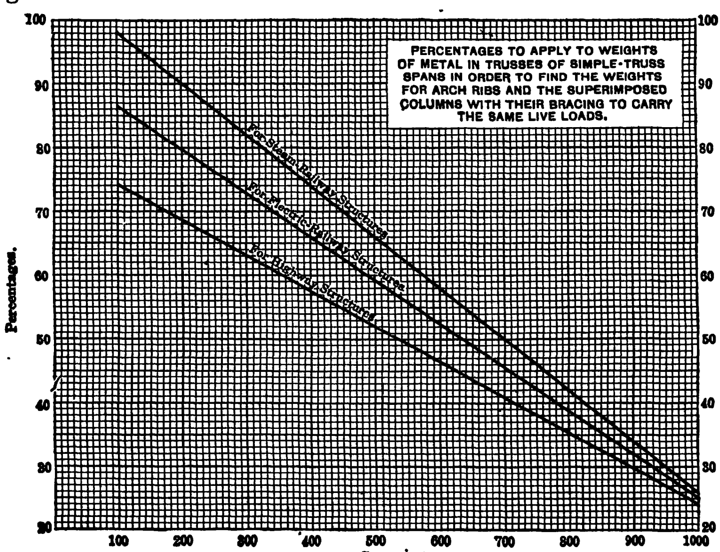


FIG. 26*d*. Percentages to Apply to Weights of Metal in Trusses of Simple-Truss Spans in Order to Find the Weights for Arch-Ribs and the Superimposed Columns, with their Bracing, to Carry the Same Live Loads.

In concluding his paper, the author quoted as follows from "Bridge Engineering":

In dealing with the comparative economics of arches and simple trusses, it must not be forgotten that there are other factors than mere weight of metal involved; for the pound price of the manufactured material is generally somewhat greater for the former, and sometimes the cost of erection also is larger. Again, the comparison of the costs of arch superstructures and truss superstructures alone is not of much importance, for

an economic investigation, to be of any value, must include both substructure and superstructure; and the costs of the former are likely to be very different in arch designs and simple-truss designs for any crossing.

Some of the solutions of the "side issues" referred to in the preceding "Synopsis" are the following:

First. The percentages to apply to weights of metal in simple-truss spans, in order to find the weights for arch ribs and the superimposed columns with their bracing to carry the same live loads, are given by the following equations:

For steam-railway structures:

$$P=106-0.08S \qquad \text{[Eq. 1]}$$

For electric-railway structures:

$$P=93-0.07S \qquad \text{[Eq. 2]}$$

For highway structures

$$P=80-0.056S \qquad \text{[Eq. 3]}$$

In these equations S is the span-length in feet, and P is the percentage to apply to the weight of metal in the trusses of any simple-truss bridge, in order to ascertain the weight of metal in the corresponding arches and the superimposed columns with their bracing. It must not be forgotten that the superior limit of S in these equations is about 1000-ft., which is as far as the recorded weights of simple-truss spans are carried, and that the inferior limit is 100-ft. In Fig. 26*d* are plotted curves giving the same information as that presented in the three preceding equations.

Second. It is evident from the computations and the resultant diagrams that the arch is more economical for highway bridges and for combined highway and electric-railway bridges than for steam-railway structures. This is because of the smaller ratio of live load plus impact to total load in the former. The larger the dead load of the flooring and floor-system, the more advantageous is it for the arch structure in comparison with the truss bridge.

Third. Based on the numerous weight computations made specially for the preparation of the paper, the following formulæ for weights of metal in the arches alone, per lineal foot of span, in arch bridges of the several types, have been established:

For Three-Hinged Arches:

$$W_a=(0.000282D+0.000426L)l \qquad \text{[Eq. 4]}$$

For Two-Hinged Arches:

$$W_a=(0.000248D+0.000416L)l \qquad \text{[Eq. 5]}$$

For Combined Two-Hinged and Three-Hinged Arches.

$$W_c=(0.000282D+0.000416L)l \qquad \text{[Eq. 6]}$$

For Hingeless Arches:

$$W_a = (0.000272D + 0.000460L)l \qquad \text{[Eq. 7]}$$

These four formulæ are based on the author's published designing specifications, which treat reversing stresses by adding one-half of the smaller stress to the larger stress and proportioning for the sum; but if the effect of reversion is entirely ignored, as some engineers deem proper, these formulæ will reduce to the following:

For Three-Hinged Arches:

$$W'_a = (0.000292D + 0.000396L)l \qquad \text{[Eq. 8]}$$

For Two-Hinged Arches

$$W'_a = (0.000258D + 0.000380L)l \qquad \text{[Eq. 9]}$$

For Combined Two-Hinged and Three-Hinged Arches

$$W'_a = (0.000292D + 0.000380L)l \qquad \text{[Eq. 10]}$$

For Hingeless Arches

$$W'_a = (0.000270D + 0.000398L)l \qquad \text{[Eq. 11]}$$

In Equations (4) to (11) inclusive, W_a, or W'_a, is the weight of metal, in pounds per lineal foot of span, in the arches of the structure; D is the dead load, in pounds per lineal foot of span; L is the live load plus impact, in pounds per lineal foot, used in making the calculations; and l is the span length, in feet. These eight equations will give fairly accurate results (slightly on the side of safety) for ordinary conditions which do not vary greatly from the theoretically economic ones.

In computing the value of L for insertion in Equations (4) to (11), inclusive, the equivalent uniform live load and the impact should be determined for the half-span-length.

Fourth. From the formulæ given above for weights of arch ribs, some interesting deductions may be drawn. For instance, in Equation (4), viz.,

$$W_a = (0.000282D + 0.000426L)l,$$

the dead load, D, is composed of the rib weight, W_a, plus the weight of floor, columns, lateral system, etc., all of which may be grouped under the symbol, W', making the equation

$$W_a = (0.000282W_a + 0.000282W' + 0.000426L)l$$

Solving this gives

$$W_a = \frac{(0.000282W' + 0.000426L)l}{1 - 0.000282l}$$

In order that W_a may be infinitely great, the divisor of the second term must be equal to zero, or

$$l = \frac{1}{0.000282} = 3{,}540 \text{ ft.} \qquad \text{[Eq. 12]}$$

This is the theoretical limiting span-length for three-hinged arches of carbon steel, or the span at which such an arch could carry nothing but its own weight without being over-stressed. It will be noted that this limiting length is the reciprocal of the dead-load coefficient in Equation (4), and that a vertical line on the diagram of arch-rib weights per lineal foot of span, drawn through the abscissa point which represents this value of l, will be asymptotic to the weight curve.

Fifth. From the curves in the diagrams it is possible to determine the economic or practicable limit of arch spans, by assuming, as the author did years ago in his economic investigations for cantilevers, that the said

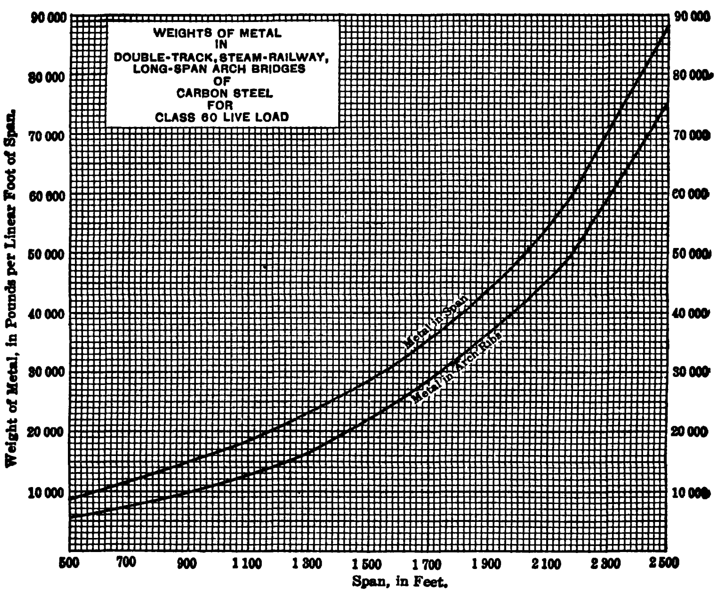


FIG. 26*e*. Weights of Metal in Double-Track, Steam-Railway, Long-Span Arch-Bridges of Carbon Steel for Class-60 Live-Load.

limit is reached when it takes $4\frac{1}{2}$ lbs. of metal to carry 1 lb. of live load. On that basis, and assuming that the equivalent uniform live load per lineal foot for a long span is equal to the car load per lineal foot, the limiting weight of metal per foot for carbon-steel, double-track, steam-railway bridges of Class 60 would be $4.5 \times 12{,}000 = 54{,}000$ lbs. Referring to Fig. 26*e*, it is found that the span for that weight is nearly 2,100 ft.; consequently, generally speaking, it may be stated that, for steam-railway arch-bridges of carbon steel, the limiting length of span is about 2,000 ft., or the same as the limiting length of main opening in cantilever bridges built of the same material. Judging by analogy, the corresponding practicable limiting length of nickel-steel arch-spans is about 2,600 ft.

Sixth. It is impracticable to take the substructure into account when

making general economic investigations for arch bridges, because no two examples thereof are alike. In case of a succession of long spans in which there is a choice of rise, a tentative layout should be made using the *anticipated* economic ratio of rise to span-length when both substructure and superstructure costs are considered, then the costs of piers and spans should

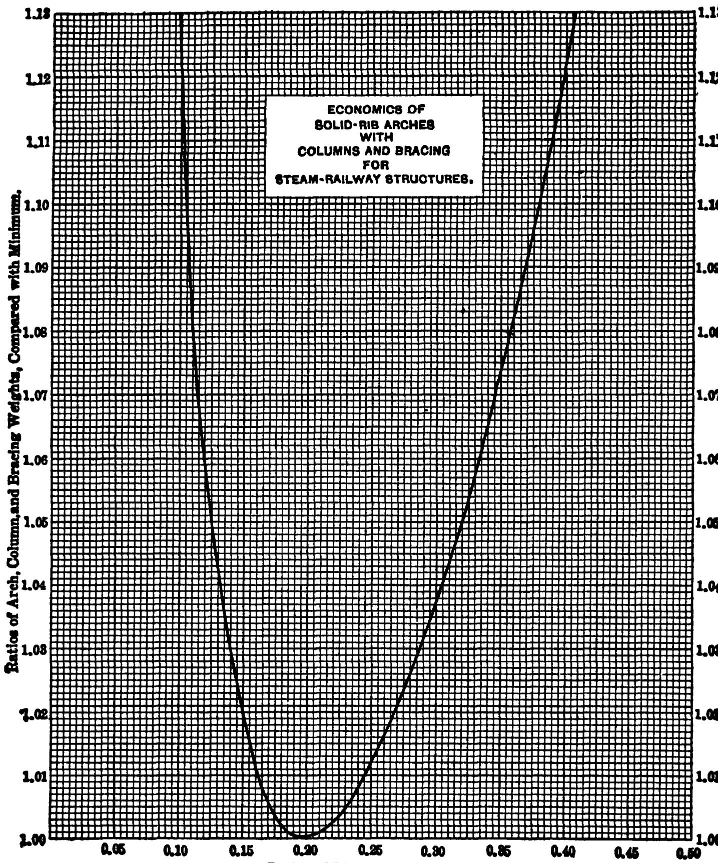


FIG. 26*f*. Economics of Solid-Rib Arches, with Columns and Bracing, for Steam-Railway Structures, in Relation to Ratios of Rise to Span-Length.

be computed, the uneconomic effect of departing from the established economic ratio of rise to span-length, given in Figs. 26*f* to 26*i*, inclusive, being employed to calculate the weights of metal from those shown in certain other diagrams.

Next, this work should be repeated for a slightly greater ratio of rise to span-length, and then for a slightly smaller ratio thereof. These three sets of computations would probably determine the best ratio to adopt;

but, if not, still another set of calculations would be necessary. The computation work involved in this investigation is not as great as a perusal of the preceding directions might lead one to surmise, especially after the computer has become accustomed to utilizing the diagrams of the paper and to estimating quickly the approximate quantities of masonry in arch piers.

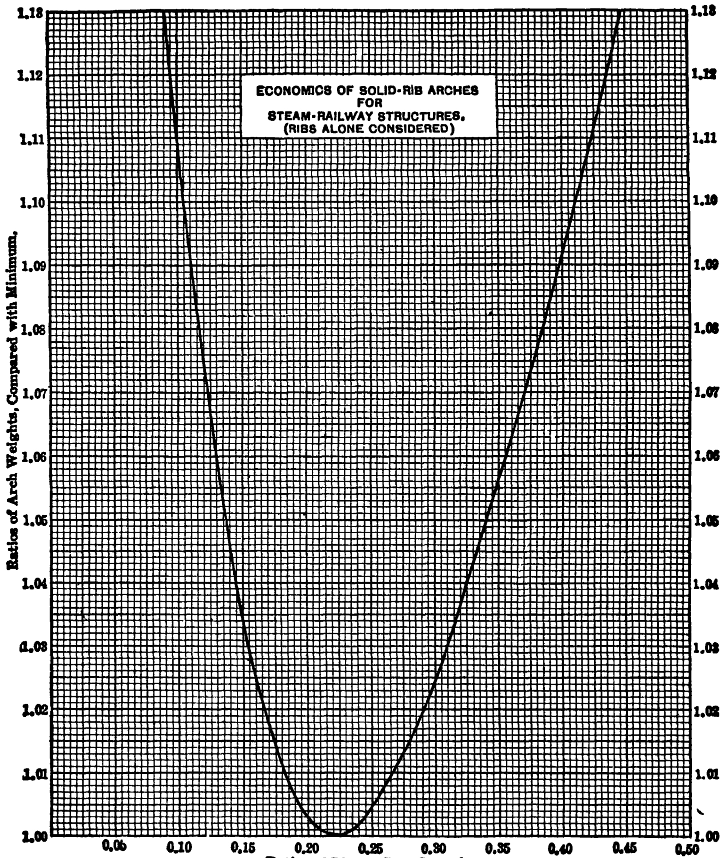


FIG. 26*g*. Economics of Solid-Rib Arches for Steam-Railway Structures (Ribs Alone Considered), in Relation to Ratios of Rise to Span-Length.

Seventh. In the cantilever arch the reduction of the horizontal thrusts at the tops of the piers, as compared with simple or non-cantilevered arches, effects a decided saving in the cost of the said piers; and the greater the proportionate lengths of the flanking spans to the arch span, the larger the economy in pier material.

Eighth. Some engineers who adhere to old-fashioned notions contend that both masonry and steel arch-bridges are applicable to bed-rock foun-

dations only, and that the piers and abutments therefor should never rest on piles. While it is true that bed-rock is the ideal foundation for such structures, pile foundations can be made entirely satisfactory, provided that the masonry of the bases is carried well down into firm material which is capable of resisting properly the horizontal thrusts. It is not legitimate to count upon the horizontal resistance of the piles; and it would be criminal practice to rest the piers and abutments of an arch bridge on piles the tops of which below the pier bases act like stilts because of passing through soft material before reaching a hard bearing.

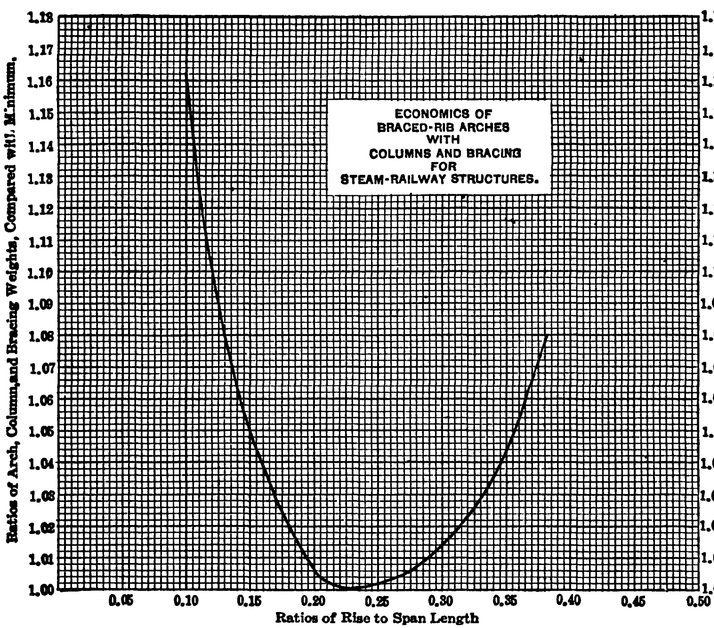


FIG. 26h. Economics of Braced-Rib Arches, with Columns and Bracing, for Steam-Railway Structures, in Relation to Ratios of Rise to Span-Length.

Ninth. As stated by Mr. F. H. Frankland in his discussion, "in respect to arch designing, the proper determination of the true economics calls for more judgment on the part of the designing engineer than with any other type of bridge."

The paper and the résumé combined contain twenty-six diagrams, of which only ten have been reproduced in this chapter; hence the reader who desires to utilize the results of that investigation, is referred to the paper itself, which, with all the discussions and the résumé, is soon to be published in the "Transactions" of the American Society of Civil Engineers.

As previously indicated, the comparative economics of steel arch-bridges

and the corresponding bridges of simple-truss or cantilever type depend greatly upon the governing conditions for the substructure, and that, as these almost invariably differ essentially at proposed crossings, it is impracticable to determine in general the economics of substructure for such com-

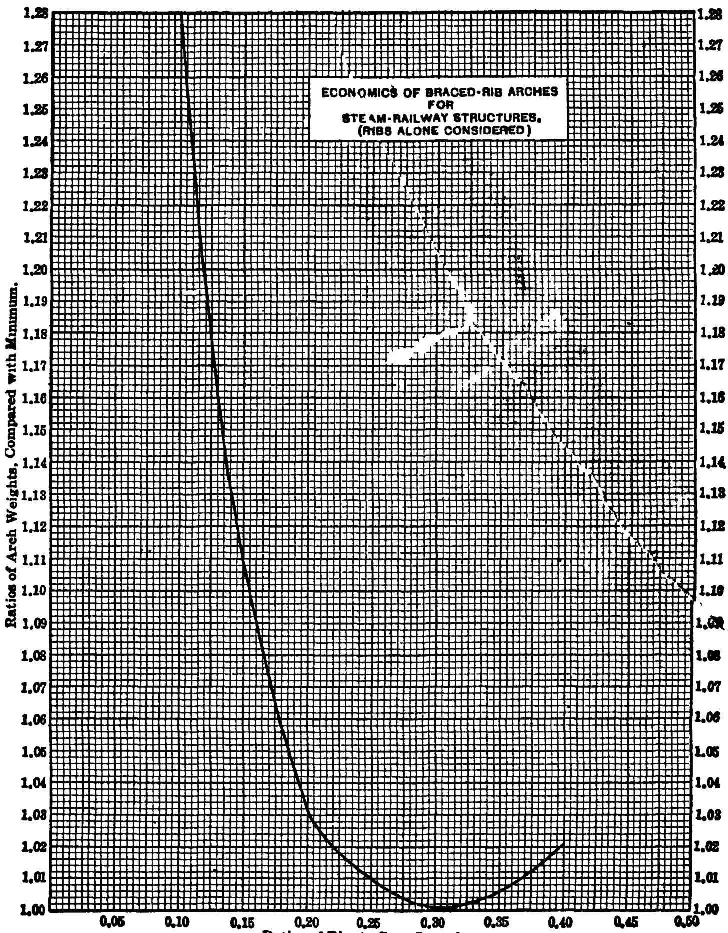


FIG. 26i. Economics of Braced-Rib Arches for Steam-Railway Structures (Ribs Alone Considered), in Relation to Ratios of Rise to Span-Length.

parisons, each case having to be treated upon its own merits. While this is certainly true, it is practicable, nevertheless, to outline some general principles that will enable a bridge designer to determine readily the suitability of any proposed crossing for an arch layout, as affected by substructure conditions; and the author offers the following:

First. Where there is a gorge with [illegible] rocky sides to be crossed by a single span, the arch is eminently suitab[illegible] not only does it reduce the

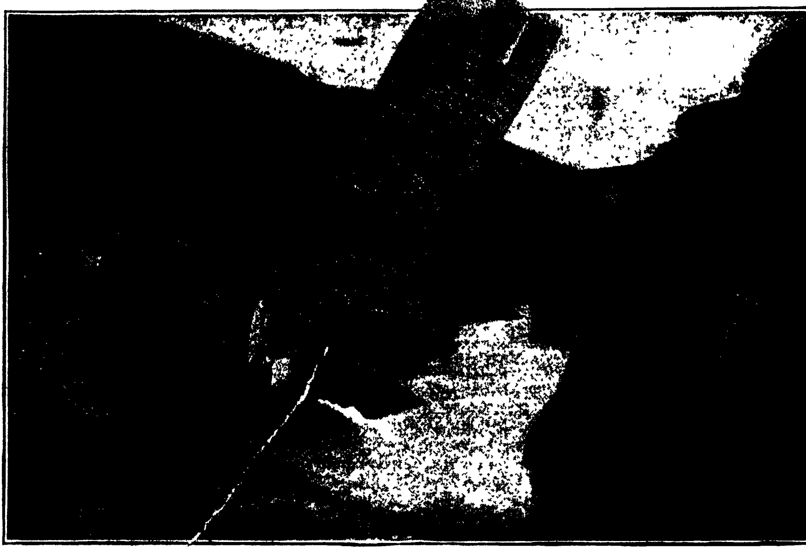


FIG 26j. Canadian Northern Pacific Railway Bridge over the Fraser River at Lytton, B C

FIG. 26k. Arch Bridge over the Waikato River at Hamilton, N. Z.

required volume of masonry to a minimum but also it brings the pressure on the rock foundations practically normal to the surface—a consideration

often of great importance when the rock would be subject to slip under vertical loading. A good illustration of such a condition is the author's Fraser River Bridge at Lytton, British Columbia, shown in Fig. 26*j*.

Second. Where there is a navigable stream to be crossed by a single span, and where the material near the water's edge is either rock or some fairly-hard material, such as shale or stiff clay, a deck truss-bridge would be inadmissible on account of navigation, and a through truss-bridge would involve high, expensive piers; hence, under these circumstances, an arch structure would be economic. Two of the author's bridges over the Waikato River in New Zealand, shown in Figs. 26*k* and 26*l*, illustrate such conditions.

FIG. 26*l*. Arch Bridge over the Waikato River at Cambridge, N. Z.

Third. Where the foundations of a wide crossing are either solid rock or some other hard material lying close to the bed of the stream or to the surface of the banks thereof, and where the grade line is much above the high-water line, a layout consisting of a succession of arch spans will frequently be economic. The springings of the arches can be brought down close to the high-water line or to the ground, as the case may be, giving small piers; whereas, for a deck simple-truss layout, much higher piers would be required. An excellent illustration of such conditions is given by the author's highway bridge over the Arroyo Seco at Pasadena, Calif., shown in Fig. 26*m*.

Another illustration of the said conditions, in which the economic advantage of the arch is not so marked, is the author's combined-highway-and-electric-railway bridge over the Colorado River at Austin, Texas, shown in Fig. 26*n*. Here the vertical distance from high-water to pier-base is considerable, thus rendering the piers much more expensive than those of the Pasadena structure.

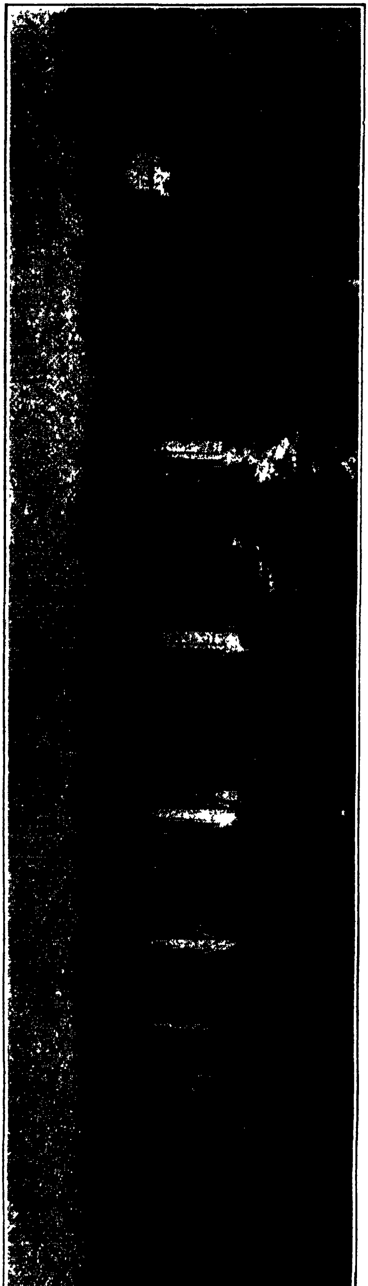


FIG. 26*m*. Arroyo Seco Bridge at Pasadena, Calif.

Where the foundation-level is much below the river-bed, the conditions are not favorable to the arch; because, in order properly to resist the horizontal thrust, the piers have to be made much wider than those for simple-truss spans.

Before concluding, it might be well to quote the following caution given by the author to those intending to utilize the information furnished in his memoir:

> The various formulæ and diagrams in this paper are to be considered as merely approximate; and though they are sufficiently accurate for preliminary estimates and for obtaining trial dead loads, they should not be used by contractors in tendering on work. The reason for this uncertainty is that the varying physical conditions at different crossings affect the arch layouts to such an extent as materially to influence the weight of metal required. As the formulæ were based on economic functions, the weights given by their use might very properly be considered as the minima possible; and any uneconomic conditions which may exist will involve an increase thereof, the amount being a matter to be determined by the computer's judgment.

FIG. 26n. Colorado River Bridge at Austin, Texas

CHAPTER XXVII

ECONOMICS OF STEEL TRESTLES, VIADUCTS, AND ELEVATED RAILROADS

THE usual railway steel trestle consists of alternate towers and intermediate spans. In determining the economic span-lengths for such a structure, there are two separate factors to be studied:

1. The ratio of length of intermediate span to that of tower-span.
2. The distance from center to center of towers.

Investigation of the first of these factors for a single-track-railway trestle shows that, for Class 40 loading, the length of tower-span should vary from 44 per cent of the distance between centers of towers for a trestle 60 feet high, to 35 per cent thereof for one 240 feet high, the length of intermediate span for these limits being from 1.27 to 1.86 times that of the tower-span. For a Class 70 loading the corresponding percentages for tower-spans vary from 46 to 38, and the corresponding span-ratios from 1.18 to 1.63. After the first factor has been investigated the second one can be determined.

Fig. 27*a* gives the economic distances from center to center of towers and the economic lengths of intermediate and tower-spans for single-track railway trestles, varying in height from 60 feet to 240 feet, and for Classes 40 to 70. Considerable variation from these economic relations will affect the total weights but slightly. It will be noted that the economic lengths are considerably greater for light loadings than for heavy ones.

Fig. 55*oo* on page 1259 of "Bridge Engineering" covers the same data as Fig. 27*a*, but the lengths there given are incorrect. In the investigation for Fig. 55*oo* no attempt was made to ascertain the economic ratio of intermediate span-length to tower span-length, it being assumed that it would be sufficiently accurate, in determining the economic distance from center to center of towers, to consider the length of the tower-span fixed, and to vary the length of the intermediate span only.

The weights of longitudinal bracing given in Fig. 55*pp* on page 1260 of "Bridge Engineering" are also incorrect. The weights should vary from 450 pounds per vertical foot for 30-foot tower-spans to 580 pounds for 60-foot tower-spans, being independent of the height.

The total weights of metal given by Fig. 55*rr* on page 1262 of "Bridge Engineering" are but slightly in error, being about 2 per cent too great for a 60-foot height, and about 6 per cent too small for a 240-foot height.

In Fig. 27*b* are given the economic span-lengths for various heights of low, single-track-railway trestles consisting of towers with two rocker

bents between, as shown in a corner of the diagram. It will be noted that, as in the previous type, the lighter the live load the greater the economic

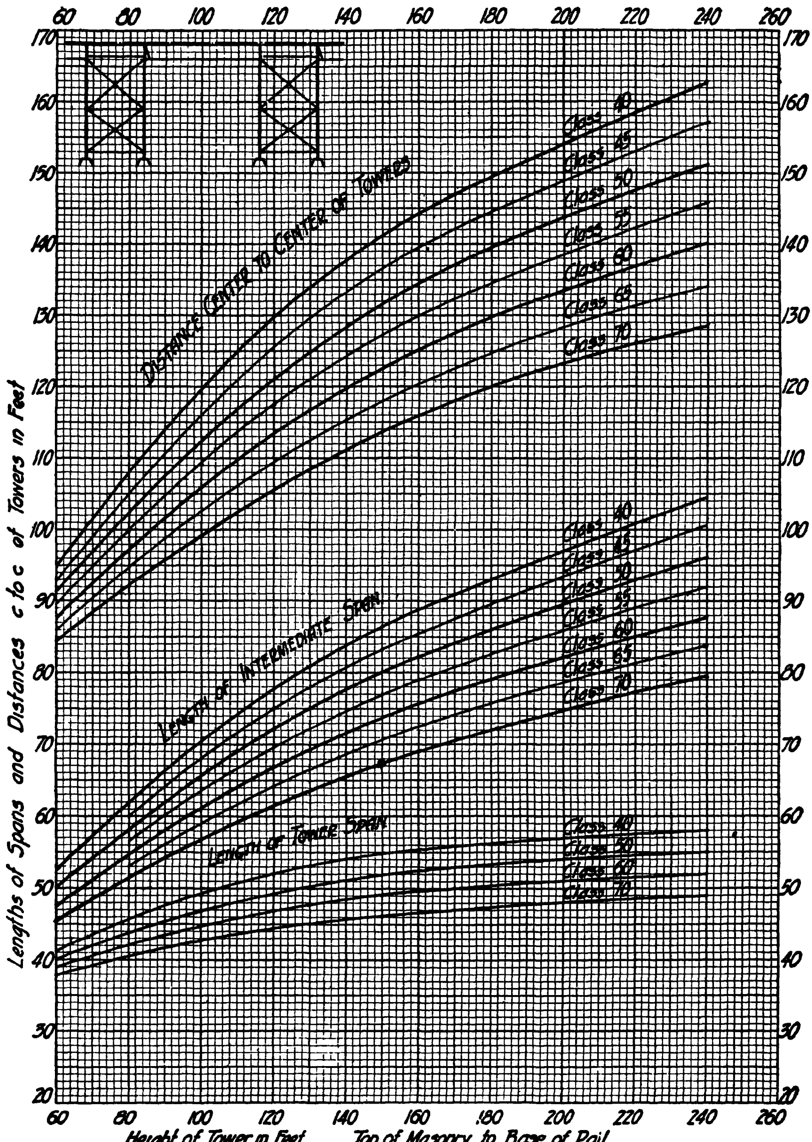


FIG. 27a. Economic Span-Lengths for High, Single-Track-Railway Trestles.

span-length. It has been assumed that the length of tower-span would in all cases be 30 feet. Actually, this length should vary somewhat, increas-

ing slightly with the height; but the resulting error in the total weights of metal will be negligible.

For double-track-railway trestles consisting of alternate tower-spans and intermediate-spans, and having two planes of longitudinal bracing, the ratio of intermediate-span-length to tower-span-length is a trifle less than for the single-track trestle, the values for Class 40 being about 1.08 for a 60-foot height, and 1.55 for a 240-foot height. The corresponding values for Class 70 are 1.05 and 1.34. The economic distances from center to center of towers are 85 or 90 per cent of those for a single-track trestle. The lengths of tower-spans are about the same as for the single-track trestles, but the lengths of intermediate-spans are only about 80 per cent as great.

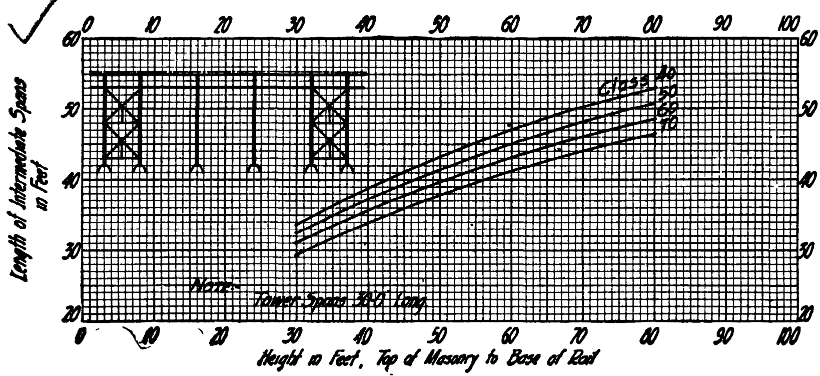


FIG. 27b. Economic Span-Lengths for Low, Single-Track-Railway Trestles.

The economics for low, double-track-railway trestles of the type shown in Fig. 27b are as follows:

The lengths of the tower-spans will be identical with those for like single-track-railway trestles, and those of the intermediate spans about ninety per cent of the corresponding lengths for same.

Strictly speaking, the costs of the pedestals should be figured in computing the economic span-lengths, but for bare rock foundations they are insignificant and, therefore, negligible; for pile foundations and spread foundations on soft soil they are almost constant per lineal foot of structure, irrespective of the span-lengths, and consequently also negligible; and it is only when the pedestals have to penetrate deeply into the ground and rest on fairly hard soil that their cost becomes influential. Their effect is to increase the economic lengths of both the intermediate and the tower spans, although probably not more than a few feet.

The economic span-lengths for highway trestles with two lines of main girders will generally be a trifle greater than those for Class 40, single-track-railway trestles; and for like structures with several lines of main girders they will be somewhat larger than those for Class 40, double-track-

railway trestles. The economic lengths will be greater for light live loads and timber decks than for heavy live loads and concrete decks.

The economics of column spacing for bents when cantilever brackets are employed is an interesting little problem, but the final determination must be in accordance with good judgment as well as economy; for if the spacing be too small, rigidity is likely to be sacrificed. Upon certain assumptions of approximate correctness the mathematical solution of this problem is a possibility; but the equations involved would be so complicated that it is much better for any particular case to assume two or three spacings, compute the total weight of metal in the bent for each, and find the one which will give approximately the least weight of metal. If the columns are placed at the quarter points of the beam, the dead-load bending-moment at the middle will be approximately zero; and if the effect of stress reversion is ignored, the direct and the reverse bending moments for the central portion of the beam will be equal, and this arrangement would be about the most economical possible. But if reversion is considered, the sectional area of the middle-portion of the beam must be greater than that of the outside portions, hence for economy its length should be somewhat less than one-half of the total, and the columns would then be spaced somewhat closer than when they are located at the quarter points. The fact that the brackets are usually lighter near the outer ends than at the inner ones would, for economy, tend to draw the columns together; but, on the other hand, this would increase the weight of the splices and connecting details. The proper column spacing to adopt will depend upon the length of the columns; for it is easily conceivable that the structure could be so high and so narrow that the quarter-point spacing would be too close for proper resistance to wind pressure. Again, in such a case the wind load might be so great as to necessitate an increase in column section above that required to care for the live and dead load stresses only; and thus the effect of wind pressure would enter the economic study. It will be found in most cases that it is inadvisable to space the columns much less than one-half of the total length of the beam.

Elevated Railroads

In respect to the economics for Elevated Railroads, as long ago as 1896 the author, when designing the Northwestern Elevated and the Union Loop Elevated Railroads of Chicago, determined certain economic functions for such structures and published the results in a paper entitled "A Study in Designing and Construction of Elevated Railroads, with Special Reference to the Northwestern Elevated Railroad and the Union Loop Elevated Railroad of Chicago, Ill.," which paper was presented to the American Society of Civil Engineers and published in its "Transactions" for 1897. From it the following statements concerning the economics of elevated-railroads have been excerpted:

Best and Most Economical Span-Lengths

This question was investigated very exhaustively, considering every item of expense, including not only the cost of metal in place, but also that of concrete, excavation, back-filling, and pavement; also the possibility of expense for the moving of water pipes and other conduits. The investigation showed that, for plate-girder construction through private property, the economic span-length is about 40 ft. for intermediate spans and 23 ft. for tower spans; while for construction in the street, where towers are inadmissible, it varies from 47 to 50 ft., or even 3 or 4 ft. more, in case of cross-girders spanning wide streets from curb to curb.

The theory of true economy in elevated railroad designing, as far as length of bays is concerned, is simply this: "The cost of the longitudinal girders should be, as nearly as may be, equal to the cost of the bents and their supporting pedestals, in cases of doubt adopting the longer span."

Four-Column *versus* Two-Column Structures

Detailed estimates of cost show that, as far as economy is concerned, there is but little, if any, difference between these two styles of bent. Whether the total cost of the four-column bent will exceed that of the two-column one for a four-track structure depends upon the various schedule prices for metal, concrete, excavation, paving, etc., as well as upon the character of the soil. As there is no great difference in the cost of these two types of structure, and as the four-column bent is decidedly the more rigid of the two, it was adopted wherever practicable. Cantilevering an entire train load beyond the exterior column of a two-column bent is not conducive to rigidity, but this is the only method that will bring the cost as low as that of the four-column bent.

Braced Towers *versus* Solitary Columns

Where an elevated railroad occupies private property and crosses the streets by spanning from curb to curb, it is practicable to use braced towers and thus stiffen the structure and check vibration; and, moreover, this arrangement is very economical.

For the Northwestern Elevated, upon which it is proposed to run trains at a speed of 40 miles per hour on the inner tracks between the inter-track stations, situated about a mile apart, the consideration of the extra rigidity afforded by the braced towers is quite important. It was therefore, decided to use both longitudinal and transverse sway bracing, forming braced towers spaced about 150 ft. apart (or two towers per block), and to use the transverse sway bracing in all bents on curves, wherever practicable.

Two only of the three spaces between columns have transverse sway

bracing, thus leaving a longitudinal passageway for wagons at the center of the structure.

The saving in weight of metal per lineal foot of four-track structure on tangents by adopting braced towers instead of solitary columns was found to be about 140 lbs., or nearly 9 per cent of the total weight.

Plate Girders *versus* Open-Webbed Girders

In designing these elevated railroads for Chicago, many estimates were made for both plate-girders and open-webbed girders, which demonstrate that there is practically no difference in the weight when both girders are properly designed. As the open-webbed girders cost a trifle more per pound to manufacture, there is no economy in their use; nevertheless they were adopted for all structures running longitudinally in the streets, in order to comply with certain city ordinances. The observance of these ordinances was sometimes carried to extremes, producing ridiculous combinations of solid and open web in the same girder, and an evident waste of material and labor. For this the engineers are not to blame, because they did not frame the ordinances.

Best Sections for Columns

Investigations concerning strength, capacity to resist impact, facility of erection, economy of metal, etc., determined that the section for columns located in the street should be two 15-in. rolled channels with the flanges turned inward and a 15-in. rolled I-beam riveted between to act as a central web or diaphragm, the flanges of the channels being held in place by interior stay plates spaced about 3-feet centers. In most cases the column feet pass below the pavement and are embedded in the concrete, to which, of course, they are bolted, but in some cases they rest on pedestals a little above the level of the sidewalk. The main object in turning the flanges inward is to enable the column better to resist impact from heavily loaded vehicles. Just above the pavement there is a curved casting filled with concrete and surrounding the column to act as a fender.

This type of column is very satisfactory after it is erected, although it gives some little difficulty in the shops and involves slightly more field riveting than usual. One complaint made was that the planes of the top and bottom flanges of I-beams are never exactly parallel to each other, hence some straightening was necessitated.

For columns located on private property or on sidewalks where the structure is transverse to the street, four Z-bars and a web plate were adopted as the most satisfactory section. At the top of the column a wide, curved web-plate and curved angles were used. This design makes a most satisfactory column, which goes through the shops readily, and which is well adapted for quick erection. It is true that it necessitated a

special tool for cutting the webs to a circular curve, but after this was made, the manufacture was easy and comparatively inexpensive.

In *Engineering News* of May 20, 1915, there appeared an excellent article by Mr. Maurice E. Griest, Assistant Designing Engineer of the Public Service Commission of New York City, entitled "Design of Steel Elevated Railways, New York Rapid Transit System," in which he gives a diagram showing that for structures located in the street the economic span-length is fifty feet, but that for lengths from forty-five feet to fifty-five feet there is not much difference in the cost. This not only confirms the author's findings of two decades earlier, but also is in accordance with a general deduction made by him of late from several economic investigations, viz., that, up to a certain limit, a material variation from absolutely economic conditions can generally be made without seriously increasing the total cost, but when the said limit is passed the uneconomics involved increases rapidly.

For further information concerning the details of elevated railroads, the reader is referred to the before-mentioned "Transactions" of the American Society of Civil Engineers.

CHAPTER XXVIII

ECONOMICS OF CANTILEVER BRIDGES

CANTILEVER bridges may be divided into four types, which cover all the layouts that are used by good designers. These are shown as Types A, B, C, and D, in Fig. 12*a*.

Type A is the one ordinarily adopted—probably as often as three times out of four. It is applicable to the case of a fairly-wide crossing, where, for some reason or other, it is not permissible or advisable to put piers in the deep water.

Type B really amounts to the doubling up of two Type-A structures by omitting the anchorages at the junction and forming the anchor arms into one continuous span. It is applicable to very wide rivers.

Type C, which is the most economic of the four, is used occasionally instead of three simple-truss spans, either for reasons connected with the navigation of the stream or because of economic motives that are sometimes based on reality but too often upon unwarranted assumption. This question is discussed at length in Chapter XII.

Type D is a combination of Types B and C, as can be seen by an inspection of the layout in Fig. 12*a*.

Comparing Types A and B, a glance at the two layouts of the diagram shows that there can be but little difference in the weights of metal per lineal foot of entire bridge; because, while the weight per foot for the anchor span is generally somewhat greater than that of the anchor arms, the entire weight of two anchorages is saved, whatever net difference there is constituting generally an excess for Type B. There can, however, be no real comparison between these two types for any particular case, as one is for a comparatively narrow crossing and the other for a very wide one.

Comparing Types A and C for a crossing in which the over-all length is fixed, but where the intermediate piers can be placed as desired, the ratio of the weight of Type C to that of Type A varies from about 0.8 for structures under two thousand feet in length to about 0.65 for structures three thousand feet long. The method of determining this may best be illustrated by an example.

Given Class 70 live load and a total length of structure of 2500′, we have from Fig. 12*a* for Type A.

$$\tfrac{5}{16}\,L+\tfrac{16}{16}\,L+\tfrac{5}{16}\,L=\tfrac{26}{16}\,L=2500' \therefore L=1538'$$

and for Type C,

$$\tfrac{11}{16}\,L+\tfrac{10}{16}\,L+\tfrac{11}{16}\,L=\tfrac{32}{16}\,L=2500' \therefore L=1250'.$$

From Fig. 55*ddd* on page 1274 of "Bridge Engineering" we find, for Class 70 and $L = 1538'$, a weight of metal per foot of 28,000 lbs.; and from Fig. 55*jjj* on page 1282 thereof, for that loading and $L = 1{,}250'$, a weight of 21,000 lbs. The ratio of these weights is $\frac{21}{28} = 0.75$.

Comparing Type D with the other types in Fig. 12*a*, it is evident that, for the same value of the hypothetical opening, L, its weight of metal is intermediate in amount; but, as before, there is really no necessity for contrasting this type with the others, because, for any crossing where it would be suitable, the other types would be wholly unsuitable.

Recapitulating, there is never any necessity for a discussion as to which of the four types should be adopted for any proposed crossing, because the profile thereof with its governing conditions will indicate clearly which is the only type applicable; but there is occasionally an economic question to determine as to whether a simple-truss layout or a cantilever layout should be adopted. This question is treated at length in Chapter XII.

In respect to the economic division of span-lengths for any proposed layout, the author determined this question for Type A nearly a quarter of a century ago when writing "De Pontibus," his findings being as follows:

First. The economic length of the suspended span is about three-eighths ($\frac{3}{8}$) of the length of the main opening, but a considerable increase or decrease of this proportion does not greatly change the total weight of metal.

Second. The most economic length of anchor arms, where the total length between centers of anchorages is given, and when the main piers can be placed wherever desired, is one-fifth ($\frac{1}{5}$) of the said total length, or one-third ($\frac{1}{3}$) of the main opening. By keeping the anchor arms short, the top chords may be built of eye-bars, provided that, with the usual allowance for impact, there is no reversion of chord stress; and this effects quite an economy of metal. But it is conceivable that cases might arise where, from danger of washout of falsework, eye-bar top chords would be objectionable; hence this method of economizing must be used with caution.

It must not be forgotten that for every dollar saved by reducing the total weight of metal through the shortening of the anchor arm, it will be necessary to spend about twenty cents for extra concrete in the anchorages. On that account, for the conditions assumed, the truly economic length of each anchor arm of a three-span, Type A, cantilever bridge may be a trifle greater than twenty per cent of the total distance between centers of anchorages.

Dr. Steinman in his "Suspension Bridges and Cantilevers," by a theoretical investigation and by using certain constants determined from computed structures, shows that for this case the length of anchor arm for economy should be four-tenths of the main opening, or four-eighteenths (0.22) of the total length of structure. This checks quite closely with the author's long-previous determination of "two-tenths or slightly more."

Third. When, however, the problem is to determine the economic length of anchor arm for a fixed distance between main piers, the result will be quite different; because, within reasonable limits, the shorter the anchor arm the smaller will be its total weight of metal, and because trestle approach is much less expensive than anchor arm. It would not, for evident reasons, be advisable to make the length of anchor arm less than twenty per cent of that of the main opening, or say fifteen per cent of the total distance between centers of anchorages. With this length there would probably be no reversion of stress in the chords of the anchor arm, even when impact is considered. Generally, though, the appearance of the structure would be improved by using longer anchor arms than the inferior limit just suggested.

Fourth. In respect to the economic length of anchor-span in a succession of cantilever spans, it may be stated that, within reasonable limits, the shorter such anchor-spans are the greater will be the economy involved; but, generally, navigation interests will prevent their being built as short as might be desired. If permissible, they may be made so short that, as in the case of anchor arms, eye-bars may be used for the top chords, thus effecting a decided economy of metal, although shortening the anchor-span increases proportionately the stresses on the web members and the weights thereof.

In regard to truss-depths for cantilever bridges, the author's practice is to make that for the suspended span, when the chords are parallel, from one-fifth of its length for short spans to one-seventh of its length for very long ones, interpolating between these limits for intermediate lengths. If one of the chords be polygonal, a greater proportionate truss depth at mid-span and a smaller one at the ends would logically be employed. The height of the vertical posts over the main piers can be made about fifteen (15) per cent of the length of the main opening, or not to exceed three and a half (3.5) times the perpendicular distance between central planes of trusses over the main piers. In the new design for the Quebec bridge these posts were made 310 feet high for the sake of appearance, although the economic length was found to be only 290 feet. These figures correspond to percentages of main openings of about seventeen (17) and sixteen (16) respectively.

There are certain legitimate economies that may be employed in the designing of cantilever bridges, among which may be mentioned the following:

A. The wind pressure assumed in computing the erection stresses may be taken lower than that given in the specifications for the finished structure, provided that the full wind pressure would not overstress any of the metal seriously or involve any risk of disaster during erection. A stress of three-quarters of the elastic limit of the metal applied a few times during erection would do no harm; and the chance of there being in such limited time any wind pressure at all approaching in magnitude that

specified is very small. This lowering of the intensity of wind pressure may be the means of avoiding, in a perfectly legitimate manner, the increasing of the sections of a number of truss members because of erection stresses; but such economizing should be done with caution after a thorough consideration of its greatest possible effects.

B. A certain amount of metal can sometimes be saved by splaying the trusses between the main piers and the ends of the cantilever and anchor arms; but unless the amount thereof be fairly large, the extra pound price of the metalwork in the cantilever and anchor arms due to the said splaying may more than offset the value of the reduction.

C. A small economy may sometimes be accomplished by omitting during erection from the cantilevered portion of the structure all parts that are not essential to its strength before the coupling of the cantilevered ends is effected, thus reducing the erection stresses a little.

D. Solitary piers or large pedestals under the main vertical posts are sometimes just as satisfactory in every way as long, continuous shafts, especially if a connecting wall of reinforced concrete between them be employed. Generally they will be found to involve a large saving in the cost of the substructure.

E. In very wide cantilever bridges it might sometimes be advisable to adopt intermediate trusses so as to economize materially in the weight of the floor-beams and sometimes a trifle in that of the trusses, also because of the consequent reduction in dead load, but mainly so as to keep within reasonable limits the sizes and weights of the pieces to be handled and thus decrease the size of the traveler and the cost of the erecting machinery. On the other hand, though, increasing the number of trusses is likely to augment a little the percentage of weight of truss details; but, where the sections of members are large, this increase would be small. In case the wind stresses are an important factor in the proportioning of the truss members, the employment of an interior truss or interior trusses might, by the reduction in areas of chord sections, cause such relatively-large wind-stresses on the chords of the exterior trusses that the additional metal required to take care of them would offset all the saving obtained in the ways just mentioned.

F. In long-span cantilever bridges the stresses on the truss members that rest upon the piers should be divided among as many such members as possible by using an inclined strut on each side as well as a vertical post, instead of carrying all the loads to the top of the latter by tension members, as was done in the original design of the ill-fated Quebec bridge. Again, if a lowering of the inner ends of the cantilever arms be permissible, the inclining of the end sections of the bottom chords to the horizontal will take up a portion of the load that is carried to the pier, and thus will reduce the stresses on the vertical and inclined posts assembling there. This last feature reduces also the total cost of the masonry by diminishing the height

of the main piers, and saves placing the tops of the trusses at an abnormal height above the water.

G. If there be any choice between the riveted and the pin-connected types of construction for any cantilever bridge, it is generally better to adopt the latter; because, as cantilever bridges are usually employed for long spans only, pin-connected work is the more suitable. Again, it is a little lighter than riveted work; and, therefore, the dead load on the structure would be somewhat less. On the other hand, the riveted construction is so much more rigid than the pin-connected that it is preferable to adopt it whenever the conditions permit; besides, in the riveted work it is not necessary to stiffen any truss members for erection, although it might be obligatory to increase a few of their sectional areas.

H. Very large compression members should be made of box section so as to do away with latticing. This not only effects an improvement in the design, but also saves some metal, although the details required at the panel points to distribute the stresses from the cut cover-plates tend to offset the saving in weight of lattice bars and stay plates.

Professors Merriman and Jacoby present in their "Roofs and Bridges," Part IV, an excellent treatment of the subject of cantilever bridges, discussed mainly from the theoretical point of view. Their economic investigations, which are based upon chord weights only, show that for a three-span cantilever of Type A, each anchor arm should be about twenty-one and two-tenths (21.2) per cent of the total length of structure. This is quite a close agreement with the twenty (20) per cent minimum found by the more accurate and practical investigation that was made for "De Pontibus." The professors find, though, thirty-nine and four-tenths (39.4) per cent of the total length of structure for the economic length of the suspended span, corresponding to about sixty-eight per cent of that of the main opening, while the "De Pontibus" investigation made it only thirty-seven and a half (37.5) per cent thereof. Actual experience has repeatedly shown that the economic length of the suspended span is from three-eighths ($\frac{3}{8}$) to one-half ($\frac{1}{2}$) of the main opening, hence the professors' figures for this portion of their work have the appearance of being incorrect; but Prof. Merriman has explained to the author by letter that he assumed the truss depth to be the same throughout the entire structure. This assumption, combined with that of ignoring the effect of the weight of the web, will account for the large discrepancy; because the professors' mathematics have been checked and found to be faultless. As a matter of fact, though, no American engineer would think for an instant of making the truss depth constant throughout the structure, because for economic reasons it should generally be about twice as great over the main piers as in the suspended span. European engineers, however, often fail to make the truss depths, especially in the cantilever and anchor arms, great enough for economy.

The professors make also from their economic investigations the following deduction: "The cantilever system hence has no theoretic economy over simple trusses when the piers can be located in any position; moreover, when the influence of the alternating stresses in the anchor arm and the material required for anchor rods are taken into account, it is at a marked disadvantage." This is true for Type A, which is the layout employed by the professors; but it is not correct in general.

The professors are right in their surmise that "probably the common three-span-cantilever bridge has a lower degree of economy than the arrangement where the simple trusses are in the end spans, as in the Kentucky River bridge"; for, as previously stated, Type-C layout requires only from eighty to sixty-five per cent as much metal as does that of Type A, for the same total length of structure. It must be remembered, however, that, as previously indicated, the comparison is hardly fair to the common three-span-cantilever, because the latter provides a greater main opening than that of the alternative layout.

CHAPTER XXIX

ECONOMICS OF SUSPENSION BRIDGES

In the designing of suspension bridges there is still much to be learned, because so few of them have been built; and this is as it should be, because they are not an economic type of structure, excepting for exceedingly long spans, or in the case of a very light highway bridge at the crossing of a gorge or a river of great depth and swift current where it would be too expensive to build piers in the stream. As shown in Chapter XIII, for steam-railway structures suspension bridges are more expensive than cantilever bridges up to the practicable limiting length of the latter; and, moreover, in respect to the important element of rigidity the former are certainly inferior. But the suspension bridge has its legitimate place in engineering construction, and that is for long-span highway bridges pure and simple, and sometimes when they carry also electric railways. There are crossings, like that of the North River at New York City, where the conditions are such as to make the use of the suspension bridge obligatory.

It is, therefore, well worth while to study the economics of the type, even to the extent of making an exceedingly elaborate investigation, as the author did lately in his memoir on "Comparative Economics of Wire Cables and High-Alloy-Steel Eye-bar-Cables for Long-Span Suspension Bridges," presented in May, 1920, to the Engineers' Society of Western Pennsylvania, of which memoir more anon.

In designing a highway suspension bridge, the first economic point to consider is that of the deck and floor-system, both of which should always be made as light as the ruling conditions will allow, because the heavier the floor the greater the load on the cables. This general question of economics in deck and floor-system has been treated in Chapter XXI, to which the reader is referred. While it is certainly advisable to cut down the dead load to a minimum, it would be anything but economic to adopt a plank base for the pavement, on account of the great danger from fire which that type of construction involves. Modern highway bridges call for a reinforced-concrete base for pavement, and there is no dodging this issue; but in suspension bridges it should be made as light as practicable by using closely-spaced stringers and thus reducing the thickness of slab to a minimum. It is true that a buckled-plate floor is lighter than a reinforced-concrete slab, but, until quite lately, as indicated in Chapter XXI, experience has shown it to be so lacking in rigidity that under the passage

of modern wheel-loads it has permitted the pavement to crack and eventually to break up. It is probable, though, that Mr. Byrne's correction of the defect will permit in future, with perfect safety, the employment of this type of flooring.

When the electric railway tracks are entirely separated from the roadways, it is economic to use the ordinary open floor of wooden ties with proper guard-rails, because the danger from fire with such a floor is not great; and even if some of the ties were to burn, no injury of any importance would be done to the metalwork. Such fires do not spread rapidly and are readily extinguished; but when a plank base with a wooden block pavement thereon catches fire on a windy day, disaster is almost sure to ensue.

There are two legitimate ways of trimming down the weight of metal in the floor-system to a minimum, viz., employing nickel steel or some other alloy for the metalwork, and supporting the cross-girders at intervals by means of intermediate trusses and cables. In adopting the first expedient, care should be taken not to reduce the depths of the joists and stringers below the limit set by good practice—otherwise their deflections would be great enough to injure the pavement. In respect to the second expedient there are both pros and cons concerning the advisability of adopting many lines of trusses and cables. The advantages are as follows:

First. The saving in weight of metal in the cross-girders.

Second. The reduction in cable cross-section due to the lightening of the cross-girders.

Third. The better distribution of load on the piers with its consequent saving of both shaft masonry and base.

Fourth. The lowering of the grade line of the structure (due to the shallowing of the cross-girders), with its consequent reduction in cost of climb for everything that crosses on the structure.

Fifth. The shortening and consequent cheapening of the approaches due to the smaller height to be surmounted.

The disadvantages are as follows:

First. The somewhat greater weight of metal due to the larger number of parts in trusses, cables, and steel towers. In long, heavy spans this really is not a disadvantage; because, on account of ease of erection, it often pays to employ comparatively light members

Second. The somewhat greater total width of deck called for to allow space for the extra trusses and cables; but this is often not disadvantageous, because great width of structure is necessitated by great span-length.

Third. In accordance with the theory of probabilities, the more numerous the trusses and cables the greater should be the total live load per lineal foot assumed for the design; for with only two lines of cables, practically the entire width of deck must be loaded in order to produce the maximum stresses in either cable, while with several lines of cables,

the loading of a much smaller width of deck will produce maximum stresses in any one cable.

The next economic consideration is the design of the stiffening trusses. It must be remembered that the weight of these is a direct function of the live load and entirely independent of the dead load, which passes immediately from the floor to the suspenders and cables without causing either moment or shear on the trusses. It is, consequently, expedient to keep the live load down to the lowest legitimate limit consistent with the probable loads from future traffic.

Custom has decreed that the truss depth should be about one-fortieth of the span length, but a greater depth would reduce the weight of metal in the chords without increasing materially that in the web. It is claimed that a greater depth than this established limit would not be æsthetic, but this can be determined only by careful study for each case as it arises.

The choice of panel length may be determined from the standpoint of æsthetics or by an economic adjustment between floor-system and trusses. This question also will require careful study.

The system of cancellation adopted for the trusses will have quite a little to do with their weights of metal, but the consideration of appearance should govern in this matter. If a single-intersection truss be adopted, it will have the advantage of avoidance of ambiguity in stress distribution, but it will not provide as pleasing an appearance as will the double system of cancellation. In the latter the vertical posts, as far as the question of statics is concerned, are superfluous; but without them the secondary stresses would run high, the connection of the floor-beams to trusses would be awkward, and the proper attachment of the trusses to the cables would be difficult. It is better, therefore, to adhere to the use of the vertical posts when the double-intersection type of truss is selected. These posts, though, may be made of minimum section consistent with their appearance and with the function which some of them perform in distributing the load to the cables.

An economic point of importance is that of having the ends of stiffening trusses free or anchored. The latter condition is more economic of metal in chords, but there is little or no difference for the web; however, it necessitates extra metal and expense for the anchoring, but the resultant effect invariably is a reduction in the total weight of metal, and hence the expedient should generally be adopted.

The selection of the versed sine for the cables is a matter of economic importance. Increasing it reduces the sectional area of cables and backstays, but augments slightly their lengths; and it adds to the height and weight of the tower columns and their bracing.

On the other hand, it effects a slight saving in mass and cost of anchorages due to the reduction of overturning moment that is caused by the diminution of stress in the backstays. Experience has shown that a depth of catenary equal to one-ninth of the span will usually give the most satis-

factory results; but there is no hard-and-fast rule about this, and it is permissible to use any depth between the limits of one-eighth and one-tenth of the span. In the past the author had never tested the economics of this feature, having been content to accept the dictum of experience as recorded in engineering text books and hand books; but in connection with the special investigations prepared for this treatise and the late economic studies upon which it is so largely based, a number of estimates have been made which have enabled him to obtain some rough figures upon the effect of increasing the depth of catenary.

The result of these computations appears to indicate that the economic depth for cables was originally adjusted upon the basis of carrying the masonry piers all the way up to the cable carriages, as in the case of the first East River bridge known as the Brooklyn Bridge; because an assumed increase in the height for this type effected no economy. But in the case of a bridge with steel towers the result was quite different; for a saving of total cost was indicated when the depth was made one-seventh of the span-length. That is probably as great a depth as a proper consideration of appearance would permit.

The question of the best type of approaches to adopt is one that has to be settled at the outset. It is an economic one in most cases, but occasionally the local conditions or the matter of æsthetics will necessitate a departure from the economic layout. Briefly, it may be stated that the type of approach which costs the least is an ordinary trestle or viaduct entirely independent of the main structure. This may be either straight or built in spiral form, as there is but little difference in the construction costs of the two, the latter generally having the advantage of saving in expense for right-of-way and property damages. Suspending the floor-system or stiffening trusses from the backstays is not economic, if it be practicable to build a trestle approach; and even if it is not, it may be better to substitute short spans for the trestle, supporting them at intervals on either piers or rocker bents, the depth of the said spans, if through ones, being made the same as that of the stiffening trusses in the case of a wire cable structure, or of any convenient or economic depth in the case of an eye-bar-cable bridge.

The uneconomics of suspending the floor of the approaches from the backstays are as follows:

First. The far greater weight of metal required for stiffening trusses and hangers as compared with that for the trestle approach, the item of pedestals for the latter being generally a bagatelle.

Second. The far greater cost of the anchorages due to the large lever arm for the overturning moment, the cable pull being horizontal and applied near the elevation of the floor.

The only case in which it is economic to suspend the floor of the approaches from the backstays is when there is deep water beneath that is required for navigation purposes. If there be fairly deep water that is

not needed for navigation, the best type of approach to adopt is a succession of deck spans of, as nearly as may be, economic length.

In the design for the anchorages there is a fine opportunity for benefiting from economic study. There are three general cases of governing conditions to consider, viz., foundations on bed rock, foundations on piles, and foundations on clay or similar material without piling. If the bed rock is fairly close to the surface, it will be advisable to found upon it; but otherwise it will be cheaper to put in shallow foundations, obtaining the necessary supporting power either by piling or by spreading the base. The maximum of economy will be obtained by making both the weight of masonry in the rear of the anchorage and the foundation area in the front thereof proportionately as large as practicable. The first expedient tends to increase the resisting moment against overturning, and the second to reduce the intensity of bearing at the toe of the face, where, of course, it is greatest. It is economic, therefore, to make the anchorages long and narrow, low in the front and high in the rear. If there be several of these in the form of walls—one for each cable, or pair of cables,—instead of one solid mass of concrete, they can be advantageously connected in front below the ground so as to spread the base, and joined in the rear by a great wall above the ground, in order to increase the weight there.

If piles be employed, they should be driven as closely together as practicable near the front of the anchorage, and, if it be found advisable, spread somewhat near the rear thereof—in other words their spacing should be adjusted to the intensity of the foundation loading. That intensity should be made as nearly uniform as possible over the entire base by the expedient just explained, thus avoiding the call for varied spacing.

With a single exception, the preceding considerations cover all the important economic features in the designing of suspension bridges. That exception is the question of the comparative economics of wire cables and high-alloy-steel eye-bar-cables, concerning which hitherto absolutely nothing definite has been known. It is one of the series of ten major economic problems in bridgework that in 1916 the author undertook to solve. Its investigation was completed in March, 1920; and the results thereof, as herein previously stated, were presented two months later, in the form of a memoir to the Engineers' Society of Western Pennsylvania. In view of the fact that the said memoir is very complete, it is here reproduced *verbatim* as follows:

Comparative Economics of Wire-Cables and High-Alloy-Steel Eye-bar-Cables for Long-Span Suspension Bridges

The economic comparison of wire cables and eye-bar cables for suspension bridges has never before been brought to the attention of American engineers, for the reason that comparatively few structures of the suspension type have been built in this country. Their unsuitability for

railway bridges, excepting those of exceedingly long span—far longer than any that have been constructed—and the slow development of the American highway system have combined to keep the type in the background; but the time is approaching when it will be necessary to carry our rapidly developing network of highways across some of the largest of our rivers, and then the suspension bridge may have an opportunity to come into its own. It must not be forgotten, however, that the conditions warranting the building of a suspension bridge are not likely to be often encountered; because it is nearly always the fiat of the War Department which necessitates a span-length so great as to render economical the building of a structure of that type.

By means of a long and elaborate economic investigation made in 1918 and published in 1919 by the Western Society of Engineers under the caption "Comparative Economics of Cantilever and Suspension Bridges," the author has shown the span-lengths of equal cost for these two classes of structure. The said lengths usually vary from 1,000 feet for highway bridges to about 2,600 feet for steam-railway bridges, the lengths for combined steam-railway and highway structures being intermediate and directly interpolated in accordance with the division of total live load, including impact allowances, between railways and highways. Later it was found that the irregularity of the abnormal unit prices of materials at present governing has lengthened the spans of equal cost some two hundred feet for highway bridges and sixty feet for steam-railway bridges; but a return to normal market conditions will assuredly bring them back to about the limits first found.

Had the original investigation been based upon the assumption of anchored ends instead of free ends for the stiffening trusses, the span-lengths for equal cost would have been found about one hundred feet shorter, or 900 ft. for highway bridges and 2,500 ft. for railway bridges at *ante-bellum* unit prices, and 1,100 ft. for highway bridges and 2,560 ft. for railway bridges at the unit prices prevailing early in 1920.

As that investigation proved that the suspension bridge is less economical for steam-railway traffic than the cantilever structure up to the extreme practicable main-span-length of the latter, and as it is well known that the cantilever is always the superior of the two in respect to the important matter of rigidity, the present investigation has been limited to the consideration of highway bridges carrying also electric railway trains, such as those that cross the East River in New York City.

The comparative advantages and disadvantages of the two types of cables are as follows:

First. The wires for the cables have higher elastic limit and ultimate strength than can probably ever be developed in eye-bars.

Second. The percentage of weight of details is far lower for wire cables than for eye-bar cables.

Third. The cost of erection is inevitably less for wire cables than

for eye-bar cables, because with the former, the process, while tedious, is not at all complicated. Before the erection of the eye-bar chains is begun, it is necessary to string across from anchorage to anchorage and over the tower tops several lines of small wire cables. These have to be used in order to carry the first few lines of bars until the latter can be made self-supporting; and although there will, of course, be some salvage on such erecting cables, their use will be quite expensive. Again, as the eye-bars will have to be placed on the pins by heating and shrinking, it is evident that the process is necessarily a slow one, requiring considerable apparatus; hence the erection cost will run high. The total cost of erection, therefore, is larger for the eye-bar cables not only because of the higher unit cost of manipulation but also on account of the greater weight of metal to be put in place.

Fourth. The lowest point in the catenary of the wire cables can be located very close to the top surface of the deck, thus making the height and the cost of the tower columns a minimum. Owing to the necessity for keeping the bottom chords of the crescent trusses above the elevation of the floor, and because, for a fair comparison, the center lines of the pairs of eye-bar cables must coincide with those of the wire cables, it is necessary to raise the tops of the towers several feet, thus increasing not only the cost of the latter but also the length and cost of the backstays.

Fifth. Wire-cable bridges do not call for special wind chords for the lateral system; but in eye-bar-cable bridges, owing to the omission of special stiffening trusses, it is necessary to provide such chords; and this item is likely to be quite expensive.

Sixth. The pound price of the wire cables is comparatively high, being at present about twice as great as that for eye-bar cables of nickel steel.

Seventh. The attachment of the wires at the anchorages is both tedious and expensive, whilst that of the eye-bars is simple and expeditious.

Eighth. The wire cables require stiffening trusses; but the two tiers of eye-bar cables are in tension under all conditions of loading; and hence they can serve, without stiffening, as the compression chords of the crescent trusses which are formed by the addition to them of vertical posts and adjustable diagonals.

Ninth. There is an uncertainty in respect to stress distribution in the stiffening trusses of the wire-cable bridge that does not exist in the eye-bar-cable structure, although, strictly speaking, the adjustable diagonals in the latter involve a small amount of ambiguity in the division of the shear. All stresses in the eye-bar-cable bridges can be determined with accuracy by the established principles of statics, whilst those in the stiffening trusses of wire-cable bridges are found approximately by several different theories based upon assumptions which are sometimes of more than doubtful accuracy; and, consequently, considerable uncertainty concerning the maximum stresses to be provided for is involved.

To as great an extent as practicable, all of these governing conditions

have been duly considered in making the economic studies for the preparation of this memoir.

When the layout of the investigation for the solution of the economic problem herein discussed was first considered, it was intended to make the said investigation comparatively short by confining it to spans of only one length and having but one live load. That span-length was 1,750 feet,—selected because it was used by the author in his preliminary study of the proposed crossing of the Delaware River between Philadelphia, Pa., and Camden, N. J., made for the Camden Bridge Commissioners.

The proposed structure consisted of a single suspension span with backstays, the approaches to it being either steel or reinforced-concrete trestlework, entirely independent of the main span. The deck, in a later modification, was laid out for a double-track electric railway at the middle, a paved roadway supported by a reinforced-concrete base twenty-two feet wide in the clear on each side thereof, and two sidewalks, each ten feet wide, cantilevered beyond the trusses, of which there were four lines. In the first design each of these trusses consisted of two eye-bar cables forming two crescents, each with a web system between composed of vertical posts and adjustable tension diagonals, but later there was also figured a wire-cable structure with stiffening trusses. In each case the towers consisted of braced steel columns with segmental-roller pedestals resting on concrete shafts supported by pneumatic caissons sunk to bed rock at a depth of about one hundred feet below low water. The anchorages were to be of plain concrete either supported by piles driven to bed rock or else resting directly on a foundation of satisfactorily-hard material, should such be encountered when making the borings. Fig. 29*a* gives the layout for the wire-cable type, and Fig. 29*b* that for the eye-bar-cable structure.

The first set of calculations prepared for this paper was based on the preceding data and consisted of five designs and estimates of cost of finished bridge upon the following lines:

First. Wire cables of very high elastic limit and ultimate strength.

Second. Mayarí-steel eye-bar-cables having an elastic limit of 50,000 lbs. per square inch and an ultimate strength of 85,000 lbs. per square inch.

Third. High-grade, nickel-steel eye-bar-cables having an elastic limit of 60,000 lbs. per square inch and an ultimate strength of 100,000 lbs. per square inch.

Fourth. Heat-treated, alloy-steel eye-bar-cables having an elastic limit of at least 75,000 lbs. per square inch and an ultimate strength of 115,000 lbs. per square inch.

Fifth. Heat-treated, alloy-steel eye-bar-cables having an elastic limit of at least 100,000 lbs. per square inch and an ultimate strength of 150,000 lbs. per square inch.

The specifications used for the designing, as far as they would apply, were those given in Chapter LXXVIII of "Bridge Engineering," and the

intensities of working stresses for the wire cables were 60,000 lbs. per square inch for wire, and either one-half of the elastic limit or one-third of the

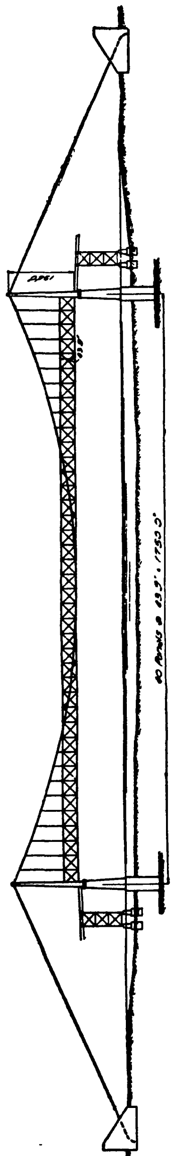


Fig. 29a. Layout of Wire-Cable Suspension-Bridge.

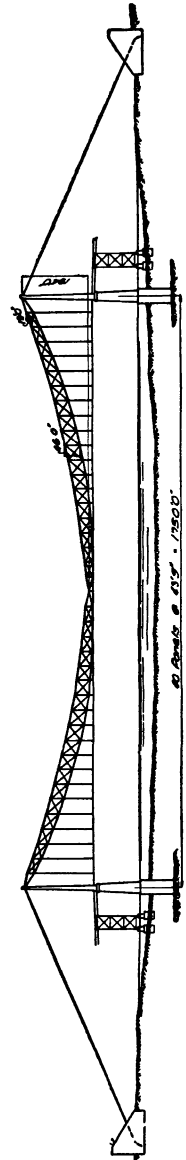


Fig. 29b. Layout of Eye-bar-Cable Suspension-Bridge.

ultimate strength for eye-bars, the lower of the two values being adopted. The material for stiffening trusses, webs of crescent trusses, floor-systems, lateral systems, towers, and anchorages was assumed to be commercial

nickel steel, having an elastic limit of 55,000 lbs. per square inch and an ultimate strength of 90,000 lbs. per square inch.

The live loads employed were as follows:

For the floor-system,

Class 25 for the electric railway,
Class B for the roadways,
Class C for the sidewalks.

For the trusses a logical combination of these loads was adopted; and a ten-car subway train was assumed on each track when figuring the stiffening trusses of the wire-cable structure and the crescent trusses of the eye-bar-cable structure. When finding the moments and shears for the former, in order to give the wire-cable bridge the best possible chance to compete, the ends of its stiffening trusses were assumed to be anchored down, although, of course, not fixed, and the theory of stress determination adopted was the approximate method given in Johnson, Bryan, and Turneaure's "Modern Framed Structures," Part II, instead of the older method of Dr. Wm. H. Burr, which, for convenience and simplicity, was taken as standard by the author in writing Chapter XXVII of "Bridge Engineering." The results of the two theories do not differ greatly, especially for ends of trusses anchored, but the latter theory requires a little less metal.

In order to ascertain the weights of metal in the crescent trusses, there was assumed for each panel point a load of unity; and its effect was computed for every web member and every chord member thereof, thus rendering it easy to find all the live load stresses and dead load stresses by means of the slide-rule. These index stresses were checked by an independent computer.

After the first set of computations was completed, the question arose as to whether the assumption of a double-deck structure carrying a much greater proportion of electric-railway live load would have caused any material changes in the results; and it was decided to repeat the calculations for a set of five double-deck structures. The result indicated no serious disagreement, as is shown in Table 29*a*.

An analysis of this table shows how very little variation there is between single-deck and double-deck bridges in respect to the proportions of the various materials in wire-cable structures and the corresponding eye-bar-cable structures. For this reason in what follows the author, with a clear conscience, has drawn his general conclusions from computations on single-deck structures only, thus saving considerable time and expense. His so doing is a good illustration of the application of the "economics of bridgework" which he is endeavoring to expound.

The investigations made up to this point permitted the establishment of a number of formulæ for quantities of materials involving variations in span length and loading, thus permitting of an extension of the study

without a seriously great augmentation of labor and expense. These formulæ are strictly logical in form. They were derived on a theoretical basis, the coefficients thus determined being changed a few per cent, where necessary, to agree with the computed weights for the various structures which had been worked out completely. It was, therefore, decided to compute and record quantities for shorter and longer spans in order to prepare curves giving, for all practicable span-lengths, the quantities of all materials *per lineal foot of main span.*

TABLE 29*a*

RATIO OF QUANTITIES OF MATERIALS IN EYE-BAR-CABLE BRIDGES TO THOSE IN WIRE-CABLE BRIDGES.

Elastic Limits of Eye-bars	Ratios							
	Cables		Nickel Steel		Anchorages and Pier Shafts		Pier Bases	
	Single Deck	Double Deck	Single Deck	Double Deck	Single Deck	Double Deck	Single Deck	Double Deck
50,000	3 76	3 75	0 73	0 69	1.09	1.08	1.16	1 15
60,000	2 82	2.82	0 68	0.66	1 03	1.02	1.06	1 08
75,000	2.05	2 05	0.65	0.62	0 99	0 98	1.00	0 96
100,000	1.42	1.41	0.63	0.60	0.94	0.93	1.00	0.88

The formulæ referred to are the following:

WIRE CABLE BRIDGES.

Stiffening Trusses.

Weight of One Chord:

Effect of Live Load Reversal Neglected,

$$W_c=0.09\frac{Ll^2}{sd}\left(0.8+4\frac{wd}{Lt}\right),\text{ but not}<0.09\frac{Ll^2}{sd}.$$

Fifty Per Cent of Reversal Stress Added to Main Stress,

$$W_c=0.12\frac{Ll^2}{sd}\left(0.9+1.5\frac{wd}{Lt}\right),\text{ but not}<0.12\frac{Ll^2}{sd}.$$

Seventy Five Per Cent of Reversal Stress Added to Main Stress,

$$W_c=0.132\frac{Ll^2}{sd}\left(0.92+1.1\frac{wd}{Lt}\right),\text{ but not}<0.132\frac{Ll^2}{sd}.$$

Weight of Web Members:

Pratt System.

Effect of Live Load Reversal Neglected,

$$W_w=0.83\frac{Ll}{s}\left(\frac{p^2+2d^2}{dp}\right)=2.5\frac{Ll}{s}\text{ for }p=d.$$

Fifty Per Cent of Reversal Stress Added to Main Stress.

$$W_w=1.25\frac{Ll}{s}\left(\frac{p^2+2d^2}{dp}\right)=3.75\frac{Ll}{s}\text{ for }p=d.$$

Seventy-five Per Cent of Reversal Stress Added to Main Stress.

$$W_w=1.46\frac{Ll}{s}\left(\frac{p^2+2d^2}{dp}\right)=4.38\frac{Ll}{s}\text{ for }p=d.$$

Warren System, Single or Double-Cancellation, with Verticals.
Effect of Live Load Reversal Neglected,

$$W_w=0.83\frac{Ll}{s}\left(\frac{p^2+1.5d^2}{dp}\right)=2.1\frac{Ll}{s}\text{ for }p=d.$$

Fifty Per Cent of Reversal Stress Added to Main Stress.

$$W_w=1.25\frac{Ll}{s}\left(\frac{p^2+1.5d^2}{dp}\right)=3.12\frac{Ll}{s}\text{ for }p=d.$$

Seventy-five Per Cent of Reversal Stress Added to Main Stress.

$$W_w=1.46\frac{Ll}{s}\left(\frac{p^2+1.5d^2}{dp}\right)=3.65\frac{Ll}{s}\text{ for }p=d.$$

Notation.

l = Main span length in feet;
d = Depth of stiffening truss in feet;
p = Panel length of stiffening truss in feet;
t = Transverse distance between chords carrying wind stresses;
L = Live load in pounds per lineal foot of truss;
w = Transverse wind load in pounds per lineal foot of bridge;
s = Working stress in tension, in pounds per square inch;
W_c = Weight of one chord in pounds per lineal foot;
W_w = Weight of web members in pounds per lineal foot of truss.

Main Cables.

Weights:

$$W=\frac{0.45l^2\sec B}{rs-0.45l^2\sec B\frac{l'}{l}}(D_s+L)=\frac{0.45l^2\sec B}{rs-0.45ll'\sec B}(D_s+L);$$

$$W\frac{l'}{l}=\frac{0.45l^2\sec B\frac{l'}{l}}{rs-0.45l^2\sec B\frac{l'}{l}}(D_s+L)=\frac{0.45ll'\sec B}{rs-0.45ll'\sec B}(D_s+L);$$

For $r/l=\frac{1}{8}$, $\sec B=1.094$, $l'/l=1.032$;

$$W=\frac{4.4l}{s-4.55l}(D_s+L);$$

$$W\frac{l'}{l}=\frac{4.55l}{s-4.55l}(D_s+L).$$

Notation.

l = Main span length in feet.
l' = Length of cable of central span in feet.
r = Rise of cable in feet.
B = Angle of inclination of cable at tower.
D_s = Superimposed dead load in pounds per lineal foot of bridge (exclusive of weight of cable).
L = Live load in pounds per lineal foot of bridge.
s = Working tensile stress in cable in pounds per square inch.
W = Weight of cable in pounds per foot of cable.
$W\frac{l'}{l}$ = Weight of cable in pounds per lineal foot of bridge.

EYE-BAR CABLE BRIDGES, FOR $\frac{r}{l}=\frac{1}{9}$.

Weight of Eye-bars in Main Span.

$$W_e=\frac{5.2l}{s-5.2l}\left[D_s+L\left(0.4+\frac{3l}{100d}\right)\right].$$

Weight of Web Members of Cable.

$$W_w=1.7\frac{Ll}{s}\left(0.8+0.2\frac{d^2}{p^2}\right),\text{ but not }<1.7\frac{Ll}{s}.$$

Notation.

l = Length of main span in feet.
r = Rise of cable in feet.
d = Maximum depth of crescent truss in feet.
p = Panel length of crescent truss in feet.
D_s = Superimposed dead load in pounds per lineal foot of bridge (exclusive of weight of eye-bars, but including weight of web members of cable).
L = Live load in pounds per lineal foot of bridge.
s = Working stress in tension in pounds per square inch.
W_e = Weight of eye-bars in pounds per lineal foot of bridge.
W_w = Weight of web members of crescent truss, in pounds per lineal foot of bridge.

The span-length selected for the low limit of the computations was 1,200 feet, and that for the high limit thereof was 2,300 feet, thus giving for all diagrams three points through which to pass each curve. Ordinarily, it is not safe to plot a curve through three points only; but in this case there were at hand several similar curves for suspension bridges established previously by the author for another economic investigation, hence no mistake was made in the plotting.

The results of the computations are shown in Figs. 29*c*, 29*d*, and 29*e*. In Fig. 29*c* are recorded on four separate diagrams the quantities of the various materials for wire-cable bridges of span-lengths varying from

1,000 feet to 3,000 feet. The curves are for single-deck structures; but the quantities found for the double-deck structures of 1,750 feet span are shown by short dotted lines. In Fig. 29*d* are indicated in a similar manner the weights of metal per lineal foot of main span for alloy-steel eye-bars in bridges of that type; and in Fig. 29*e* are recorded correspondingly the

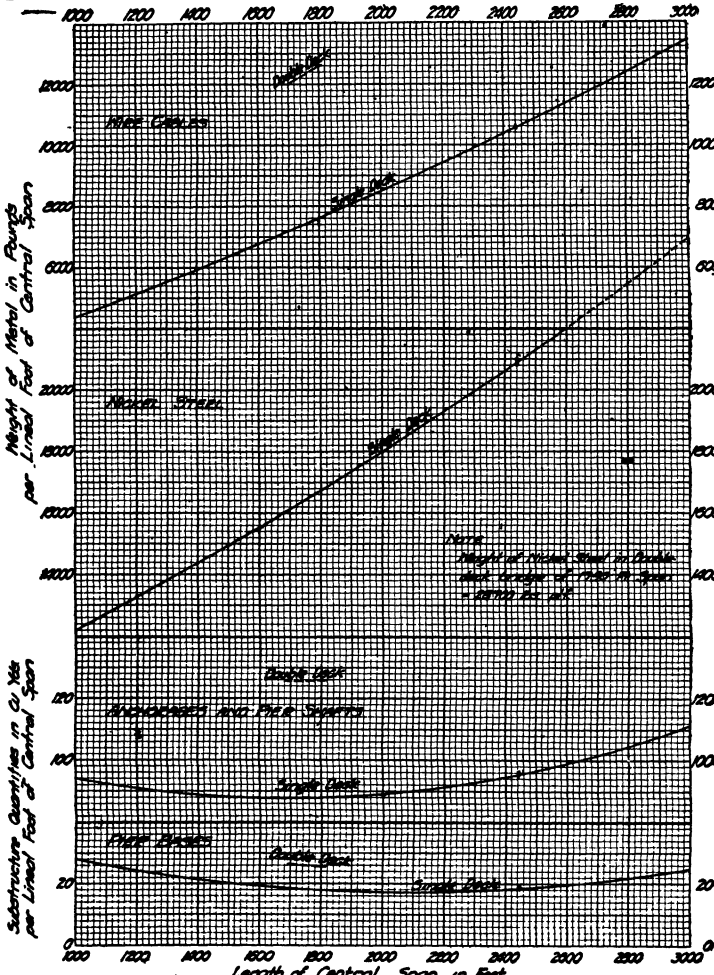


FIG. 29c. Quantities for Wire-Cable Suspension-Bridges.

quantities of nickel steel, concrete in shafts and anchorages, and mass of materials in pier bases for such structures.

In order to illustrate the manner of using these diagrams and at the same time to draw therefrom conclusions concerning the present status of the economics of the two types of cable under discussion, it will be well to

solve several examples based on current unit prices of the materials in place. These may be taken as follows:

Wire Cables........................	23¢ per lb
Mayarí steel.........................	10¢ "
Commercial nickel steel..............	11¢ "
High-grade nickel steel...............	12¢ "
Concrete in pier shafts and anchorages..	$16.00 per cu. yd.
Mass of pneumatic bases.............	$35.00 " "

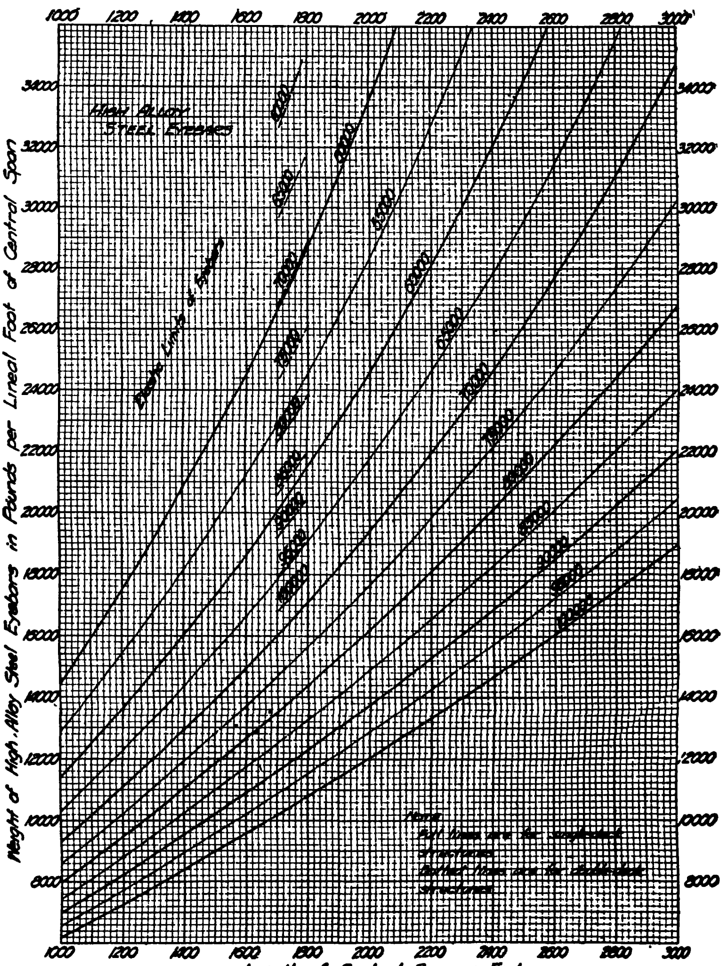


FIG. 29*d*. Weights of Alloy-Steel Eye-bars in Eye-bar-Cable Suspension-Bridges.

In pitting the eye-bar-cables of the various alloy-steels against wire cables it will suffice to figure for three spans, viz., those of 1,000, 2,000, and

3,000 feet. Attention is called to the fact that the costs found per foot of main span are not total for the structures; because the items of costs of deck and floor-system, being the same for both types, have been omitted for the sake of simplicity. The lateral system, though, has been taken

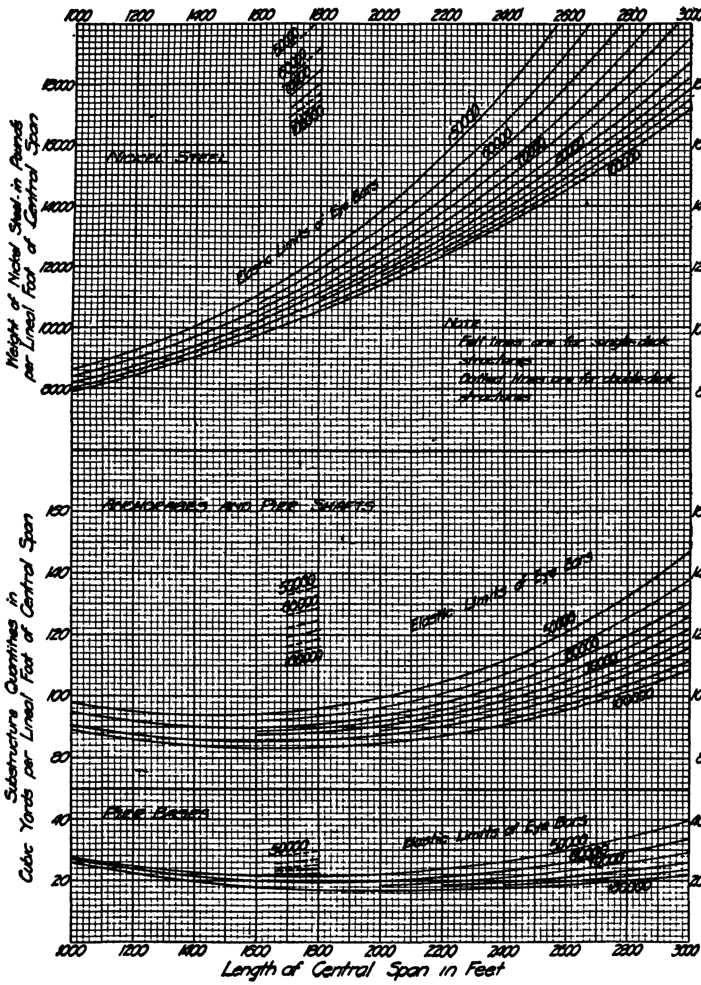


FIG. 29e. Quantities of Various Materials in Eye-bar-Cable Suspension-Bridges.

into account, because the weights of metal thereof in the two types of structure differ materially. The principal reason for this is that, in the eye-bar type, special wind chords will be obligatory; whilst, in the wire-cable type, the chords of the stiffening trusses serve as wind chords, only a small increase in sections above those needed for live loads being required.

Mayarí Steel Comparison.

1,000-Foot Span.

From Fig. 29*c* we can make for the wire-cable structure the following estimate:

Wire cables........	4,300 lbs.	@	23¢ =	$989.00
Nickel steel........	12,200 "	@	11¢ =	1,342.00
Plain concrete......	94 cu. yd.	@	$16.00 =	1,504.00
Mass of bases......	28 "	@	$35.00 =	980.00
Total			=	$4,815.00

From Figs. 29*d* and 29*e* we have for the Mayarí steel the following estimate.

Mayarí steel.......	14,400 lbs.	@	10¢ =	$1,440.00
Nickel steel........	8,600 "	@	11¢ =	946.00
Plain concrete......	98 cu. yds.	@	$16.00 =	1,568.00
Mass of bases......	28 "	@	$35.00 =	980.00
Total			=	$4,934.00

2,000-Foot Span.

WIRE-CABLE STRUCTURE

Wire cables........	8,500 lbs.	@	23¢ =	$1,955.00
Nickel steel........	18,000 "	@	11¢ =	1,980.00
Plain concrete......	89 cu. yds.	@	$16.00 =	1,424.00
Mass of bases......	17 "	@	$35.00 =	595.00
Total			=	$5,954.00

EYE-BAR-CABLE STRUCTURE

Mayarí steel.......	33,600 lbs.	@	10¢ =	$3,360.00
Nickel steel........	13,800 "	@	11¢ =	1,518.00
Plain concrete......	100 cu. yds.	@	$16.00 =	1,600.00
Mass of bases......	22 "	@	$35.00 =	770.00
Total..........................			=	$7,248.00

From these four estimates it is evident that at present prices untreated Mayarí-steel eye-bar-cables cannot compete with wire cables in suspension bridges. With the shortest economic span-length for highway suspension bridges, viz., 1,000 feet, in order to compete with wire, the untreated Mayarí steel would have to be put in place at a pound price of less than 9.2 cents. While this could probably be done without actual loss, it can readily be seen that, in general, untreated Mayarí-steel eye-bars cannot be employed economically for suspension-bridge cables.

As heat-treated, carbon-steel eye-bars have an elastic limit of 50,000 lbs. per square inch, the same as that for untreated Mayarí-steel eye-bars, and as the unit price erected is also about the same, the conclusions reached concerning the latter will apply also to the former.

As for heat-treated, Mayarí-steel eye-bars, the author has no data regarding their strength, nor is he convinced that the irregularity in the elastic limit of that alloy would not militate too strongly against using it in bridgework after heat treatment. Judging, though, from the computations which follow concerning heat-treated nickel-steel eye-bars, one would surmise that suspension bridges of heat-treated, Mayarí-steel eye-bars might be economical up to spans of 2,000 feet.

HIGH-GRADE NICKEL-STEEL COMPARISON.

1,000-Foot Span.

EYE-BAR-CABLE STRUCTURE

Eye-bar cables.....	11,400 lbs.	@	12¢	=	$1,368.00
Nickel steel........	8,300 "	@	11¢	=	913.00
Plain concrete......	94 cu. yds.	@	$16.00	=	1,504.00
Mass of bases......	27 "	@	$35.00	=	945.00
Total......................				=	$4,730.00

2,000-Foot Span.

Eye-bar cables.....	24,600 lbs.	@	12¢	=	$2,952.00
Nickel steel........	12,800 "	@	11¢	=	1,408.00
Plain concrete......	94 cu. yds.	@	$16.00	=	1,504.00
Mass of bases......	20 "	@	$35.00	=	700.00
Total......................				=	$6,564.00

A comparison of these figures with those previously made for wire-cable structures shows that the high-grade, untreated, nickel-steel is economic for 1,000-ft. spans but not for those of 2,000 feet, hence let us test for an intermediate length.

1,500-Foot Span.

WIRE-CABLE STRUCTURE

Wire cables........	6,400 lbs.	@	23¢	=	$1,472.00
Nickel steel........	15,000 "	@	11¢	=	1,650.00
Plain concrete......	88 cu. yds.	@	$16.00	=	1,408.00
Mass of bases......	20 "	@	$35.00	=	700.00
Total......................				=	$5,230.00

EYE-BAR-CABLE STRUCTURE

Eye-bar cables.....	17,300 lbs.	@	12¢	= $2,076.00
Nickel steel........	10,100 "	@	11¢	= 1,111.00
Plain concrete......	90 cu. yds.	@	$16.00	= 1,440.00
Mass of bases......	20 "	@	$35.00	= 700.00
Total......................				= $5,327.00

From these figures it is seen that untreated, high-grade, nickel-steel eye-bars can compete today with wire cables for span-lengths below about 1,400 feet. If they were heat-treated, their intensities of working stress could probably be made as high as 40,000 lbs. per square inch, corresponding to an elastic limit of 80,000 lbs. or an ultimate strength of 120,000 lbs. The value of these eye-bars in place would be 13 cents per lb.

Heat-Treated Nickel-Steel Comparison

2000-*Foot Span.*

EYE-BAR-CABLE STRUCTURE

Eye-bar cables......	16,100 lbs.	@	13¢	= $2,093.00
Nickel steel.........	11,900 "	@	11¢	= 1,309.00
Plain concrete.......	87 cu. yds.	@	$16.00	= 1,392.00
Mass of bases.......	18 "	@	$35.00	= 630.00
Total........................				= $5,424.00

This is considerably less than the cost previously found for a wire-cable structure, consequently let us test for a 2,500-ft. span.

2500-*Foot Span.*

WIRE-CABLE STRUCTURE

Wire cables.........	10,800 lbs.	@	23¢	= $2,484.00
Nickel steel.........	21,400 "	@	11¢	= 2,354.00
Plain concrete.......	97 cu. yds.	@	$16.00	= 1,552.00
Mass of bases.......	19 "	@	$35.00	= 665.00
Total........................				= $7,055.00

EYE-BAR-CABLE STRUCTURE

Eye-bar cables......	21,000 lbs.	@	13¢	= $2,730.00
Nickel steel.........	14,800 "	@	11¢	= 1,628.00
Plain concrete.......	98 cu. yds.	@	$16.00	= 1,568.00
Mass of bases........	20 "	@	$35.00	= 700.00
Total........................				= $6,626.00

From these assumed figures it appears that heat-treated, high-grade, nickel-steel eye-bars could probably be used economically for suspension-bridge spans up to at least 3,000 feet.

It is within the realm of possibility that in a few years there will be manufactured heat-treated, chrome-molybdenum-steel eye-bars having an ultimate strength of 150,000 lbs. per square inch, for which the intensity of working stress may be taken at 50,000 lbs. per square inch, corresponding to a minimum elastic limit of 100,000 lbs.; and that the metal in place will be worth not to exceed 15 cents per lb.

Let us test this for a 3,000-ft. span.

Heat-Treated Chrome-Molybdenum-Steel Comparison

3000-*Foot Span.*

WIRE-CABLE STRUCTURE

Wire cables	13,400 lbs.	@ 23¢	=$3,082.00
Nickel steel	25,000 "	@ 11¢	= 2,750.00
Plain concrete	112 cu. yds.	@ $16.00	= 1,792.00
Mass of bases	25 "	@ $35.00	= 875.00
Total			=$8,499.00

EYE-BAR-CABLE STRUCTURE

Eye-bar cables	19,000 lbs.	@ 15¢	=$2,850.00
Nickel steel	17,200 "	@ 11¢	= 1,892.00
Plain concrete	110 cu. yds.	@ $16.00	= 1,760.00
Mass of bases	22 " "	@ $35.00	= 770.00
Total			=$7,272.00

From these figures it is evident that the hypothetical "Chromol" steel at the hypothetical pound price used would be much more economical than wire for suspension bridges of all possible span lengths.

Résumé of Findings

Summarizing the results of the entire investigation, on the basis of present unit prices, the following conclusions are reached.

First. Neither untreated Mayarí-steel eye-bars nor heat-treated carbon-steel eye-bars can compete with wire in the building of highway suspension bridges.

Second. If Mayarí-steel eye-bars after being heat-treated are reliable and satisfactory, it is not unlikely that they can compete with wire cables for spans up to 2,000 feet.

Third. Untreated eye-bars of high-grade nickel-steel are more economic than wire for spans up to 1,400 feet.

Fourth. Heat-treated eye-bars of high-grade nickel-steel are probably economic for all spans less than 3,000 feet.

Fifth. If satisfactory eye-bars can be made of heat-treated "Chromol" steel, and having an elastic limit exceeding 100,000 lbs. per square inch and an ultimate strength of not less than 150,000 lbs. per square inch, they will be more economical than wire cables for suspension bridges of any feasible span-lengths.

CHAPTER XXX

ECONOMICS OF MOVABLE SPANS*

In dealing with the economics of movable spans it will suffice to consider only those types thereof which have survived the test of time, relegating the others to oblivion. The description and history of all such types, good, bad, and indifferent, will be found in Chapters XXVIII to XXXI, inclusive, of "Bridge Engineering." As stated there, the surviving types are the swing, the bascule, and the vertical lift; and the first mentioned, as will be shown further on in this chapter, has no longer any real *raison d'être*. The choice today, consequently, is between the bascule and the vertical lift, with the preponderance of advantage and economy in most cases favoring the latter.

Before beginning a discussion of the comparative costs of the three surviving types, it will be well to consider thoroughly all their important advantages and disadvantages, excepting only those that relate to first cost of construction, plus capitalized cost of maintenance and repairs.

Swing-Span versus Either Bascule or Vertical Lift

First. The swing provides two openings, while either the bascule or the vertical lift affords only one. This is claimed by the advocates of the swing as an advantage; but it is not often such, because very seldom is there a location at which there exists a possibility of the water traffic being so great as to necessitate the simultaneous passage of vessels in opposite directions, or such a large amount thereof in one direction as to call for two openings. Probably not one location in a hundred would have so many craft passing that two openings would be utilized at the same time, excepting semi-occasionally. But if such were the case, the single opening could be enlarged so as safely to permit two vessels to pass at the crossing. The question would then arise as to how greatly the single opening should be increased in order to afford equal facility for passing, as compared with a structure having two openings. In the author's opinion, if the single opening in ordinary cases were made twenty-five per cent wider than either opening of the swing, the facility thus provided for the simultaneous pas-

* This chapter was presented as a memoir to the American Railway Engineering Association in Dec., 1920, and is now due to appear in its "Proceedings." It will be submitted to the leading bridge engineers of this country (both in and outside of the Association) for a thorough discussion.

sage of two vessels would not be inferior; but for small openings, of course, this percentage of increase would have to be much greater.

Second. The swing, on account of either its pivot pier or its draw protection, offers much more obstruction to the flow of water than does either of the other types.

Third. The cost of maintenance is more in a swing span than in either of the others on account of the upkeep and periodical replacement of a costly and perishable drawn protection.

Fourth. The swing, of necessity, occupies space outside of that required for the accommodation of land traffic, while the other types do not.

Fifth. The least practicable time of operation is usually twice or thrice as great for a swing as for a corresponding single-leaf bascule or vertical lift.

Sixth. Either the vertical lift or the single-leaf bascule affords better automatic adjustment of the railroad tracks thereon than does the swing span.

Seventh. In the case of future enlargement of bridge to accommodate an increase of traffic, the swing has to be torn down and rebuilt, but a vertical lift or a bascule can simply be duplicated alongside.

Eighth. The danger of the span's being struck, when in motion, by passing vessels is much greater in the case of a swing than in that of either of the other types.

Ninth. The wider the roadway of a swing the more obstructive does it become to navigation, whilst the widening of either a vertical lift or a bascule does no harm thereto whatsoever.

Tenth. In passing vessels with low masts, a swing has to open just as fully as for a high-masted craft, which is not the case with a vertical lift or a bascule.

Eleventh. In sand-bearing streams the protection-works for the moving span of a swing bridge cause a deposit of sediment, and thus often tend to obstruct navigation.

Twelfth. In the case of a shifting channel, the two openings in a swing may score an advantage for that type over the other types, in that vessels might be able to pass through one opening after the other has been silted up; but under such conditions the silting is more than likely to block both openings. Moreover, for such conditions the vertical lift is far superior to the other two types, in that the design of the structure can be made so as to raise at any time any one of several similar spans, simply by shifting thereto the towers, the machinery, and the house or houses.

Vertical Lift versus Bascule

Comparing the vertical lift with the bascule, the former has several advantages, amongst which may be mentioned the following:

First. The floor is always horizontal, permitting the employment of

any type of deck that can be used on a fixed span, which is not the case for a bascule. That type of movable span necessitates a timber deck with its consequent fire-risk.

Second. Great wind pressure during operation has no appreciable effect on a vertical lift, while it may cause serious delay to a bascule, or even, under extreme conditions, prevent its operation altogether.

Third. The vertical lift does not have to rise so high for low-masted passing craft as does the bascule; and thus it saves a considerable amount of time and power.

Fourth. In railroad bridges when the moving span is down, it acts just like any fixed span, as far as operation under traffic is concerned, which cannot be said for either the swing or the double-leaf bascule; or, in other words, for railroad traffic the vertical lift is the most rigid of the three types, excepting only in the case of the single-leaf bascule, which is usually quite rigid.

Fifth. In case of a shifting channel, it is feasible to make a number of the spans alike and to arrange, for any time in the future and at comparatively moderate expense, to have the towers and machinery taken down, transferred, and re-erected, so as to lift any one of the said like spans. This could not by any possibility be done in the case of any other type of movable structure.

Sixth. The vertical lift, when its towers do not rest on flanking spans, lends itself readily to a future raising or lowering of the grade line in a way that no other type of movable span can possibly do; for all that is necessary is to change the elevation of the bearings of the lift span. If a change of grade be anticipated when the plans are being prepared, provision should be made therefor by increasing adequately the heights of the towers; but if at any time the grade on a vertical-lift bridge of the type mentioned, for which no such preparation has been made, has to be raised to such an extent that there will be interference because of the counterweights reaching the new decks of the approaches in the towers, the result desired could be accomplished by arranging for a small portion of the said approaches to move either laterally or vertically out of the way of the counterweights, whenever a very-tall-masted vessel has to pass. For any other vessel, however, these moving approaches would not have to be utilized; consequently, they would seldom need to be operated.

Seventh. The vertical lift accommodates itself to a skew crossing far better than does the bascule, as in the latter the tail has to be squared, while in the former both the span and the towers may be skewed, thus reducing the clear waterway (and consequently the length of moving span) to a minimum.

Eighth. By spanning the opening between tops of towers in a vertical-lift bridge, electric-wires, water-pipes, and gas-pipes can be carried across; but the accomplishment of this in the case of a bascule or a swing would

necessitate either expensive and troublesome submarine cables and conduits or special towers for carrying an overhead span.

Ninth. The inherent simplicity of the vertical lift as a piece of mechanism, compared with the bascule, makes it more reliable in operation, and, on that account, somewhat less expensive. For this reason the vertical lift would have an advantage over the bascule in many foreign countries, such as those of Latin America, where the conveniences for repairing or replacing parts are not close at hand.

Erection requirements or other special conditions at a site often affect materially the relative economics of the various types of movable spans—sometimes to such an extent as to outweigh all other considerations. They may affect either favorably or unfavorably any of the different types. Any given site should, therefore, receive special study from this viewpoint.

Economics of Swing Spans

Although the author does not believe that there is to-day any necessity for this type of structure nor any advantage to be derived from building one; yet, as all engineers may not agree with him, it will be well, in order that this chapter may not be lacking in completeness, to give a short dissertation concerning the economics of some of the different types of swing in common use.

Rim-Bearing versus Center-Bearing Spans

The choice between these two types is mainly a matter of taste or sometimes one of prejudice; for there is no great difference in their first costs, what there is being in favor of the latter, which also has a slight advantage in respect to amount of power required to operate. In the author's opinion, the principal economic advantage of the center-bearing type is due to the smaller diameter of the pivot pier.

There is a difference of opinion amongst railroad engineers, and even amongst high authorities on bridges, concerning both the relative merits and the economics of these two types. The late C. C. Schneider, Past President of the American Society of Civil Engineers, said: "The center-bearing type, designed in accordance with good modern practice, offers more advantages than the rim-bearing type, and should always receive the first consideration in determining upon a design. It requires less power to turn, has a smaller number of moving parts, is less expensive to construct and maintain, involves less accurate construction than the rim-bearing bridge, and does not as easily get out of order. The structural and the operating-machinery parts are entirely separate; and when the bridge is closed, it forms either two independent fixed spans, or a fixed span, continuous over two openings, resting on firm, substantial supports.

There are no ambiguities in the calculations in reference to the distribution of the load; and the distance required from base of rail to masonry is generally less than that required for a rim-bearing bridge with proper distribution of the load over the drum. Any irregular settlement of the masonry does not materially affect its operation.

"On the other hand, the rim-bearing bridge requires a circular girder or drum of difficult and expensive construction, a ring of accurately-turned rollers, and circular tracks that necessitate great care in their construction and delicate adjustment in their erection, in order to make the bridge operate satisfactorily. Repairs are troublesome and expensive; and any irregular settlement of the masonry will throw the whole turning apparatus out of order."

As an illustration of diametrically opposite opinion, the following quotation from a printed statement by the late C. H. Cartlidge, formerly Bridge Engineer of the Chicago, Burlington, and Quincy Railway, is directly to the point. "The writer's experience with center-bearing draw-spans has been such as to prejudice him against them for spans of any magnitude. It seems difficult at any reasonable cost to proportion the pivot-bearing so that it will not wear; and any wear on a pivot-bearing is expensive to repair. On one draw the wearing away of the bronze bearing in the pivot allowed the upper and lower castings to rub, making the turning of the draw a very noisy operation, while the few wheels provided to steady the span during turning were overworked and cut the circular track badly."

There is a combination of the rim-bearing and the center-bearing swings advocated by some engineers; but the author, on general principles, objects to hybrid designs, and especially in this case where there must exist a great uncertainty concerning the distribution of load between rim and pivot.

Bob-Tailed Swing versus Ordinary Swing

While there is apparently a saving in first cost by cutting down the length of one arm of a swing-span so as to convert it into a "bob-tailed" structure, that saving is generally absorbed by the adoption of more power and heavier machinery (with which to operate against unbalanced wind loads), heavy counterweights, and the special metal needed to support the said counterweights.

There are other questions of economics in swing spans, such as plate-girder *versus* truss-span structures, and continuous *versus* non-continuous trusses over pivot piers; but in view of the fact that the author is opposed on principle to building any more swing spans, it were useless to carry further this economic dissertation, especially as the subject of swing bridges is treated quite thoroughly in Chapter XXIX of "Bridge Engineering."

Economics of Bascule Spans

Bascule spans may be divided into two general classes—single-leaf and double-leaf. The former type is superior to the latter in rigidity but inferior in appearance, because of lack of symmetry. In the opinion of most railway engineers, on account of the difficulty in properly connecting the outer ends of the two leaves, the double-leaf bascule ought not to be employed for steam-railway bridges, for the reason that the lack of rigidity and the great deflection involved are not compatible with truly-first-class construction.

There is an economic question in connection with bascules that is very difficult to solve, viz., what is the distance between centers of bearings at which it will save in first cost to change from a single-leaf to a double-leaf structure? In the case of a bridge in which the counterweights, the machinery, and their supporting metal are below the deck, the economic limit for the single-leaf span will almost always be less than it is when those parts of the structure are above; and, in the former, the closer the deck is to high-water level, the shorter will be this limiting economic distance. The reason for this is that, with a single-leaf structure and a small vertical distance between grade and high water, unless the moving span be short, either the counterweight will be excessively heavy, or else a pit will have to be provided to receive the tail end. The adoption of either of these expedients causes the cost of structure to rise rapidly.

One of the reasons why the cost of a two-leaf bascule tends to exceed that of the corresponding one-leaf structure is that in the former there must be a holding-down reaction at each end; and because that reaction involves the use of considerable extra metal in the flanking spans or over the piers—much of it being high priced. Since this anchorage is required for live load only, it follows that the condition of small live load and large dead load favors the double-leaf bascule, whereas that of large live load and small dead load favors the single-leaf type.

With counterweights, towers, and machinery above the deck, the clear opening for equal cost of one-leaf and two-leaf spans is probably so great as to exceed the length above which it becomes economic to pass from bascule to vertical lift. While the author has made no special figures to establish beyond all doubt the correctness of this statement, his experience with bascule designing warrants him in drawing the conclusion. He is of the opinion that, for the overhead-counterweight type, the length for equal cost lies between one hundred and fifty and two hundred feet; and, for such a span-length, the vertical lift, for the sake of economy, if for no other reason, should supplant the bascule.

It is also the author's opinion, based on practical experience rather than upon extensive special economic computations, that the double-leaf type of bascule is necessitated only by reason of æsthetics or because of a

too limited vertical distance between the elevations of grade and high water.

The simplest form of bascule is the ordinary heel-counterbalanced, trunnion type; and this is the kind which is generally adopted when the minimum clearance allowed above water will permit. In many cases the height is not sufficient for the heel of the span and the counterweight to clear the water or the pier-tops, and then the span must be lengthened and a water-tight pit must be provided into which the said heel and the counterweight may descend. The expense of construction thus involved is very great; and, consequently, the more complicated and unsightly types, having towers and counterweights above the roadway, are resorted to.

Sometimes cases occur in which the height above water is insufficient for a simple, heel-balanced bridge of the ordinary type without water-tight pits, and where the adoption of unsightly towers and counterweights is barred for æsthetic reasons. Again, in the usual heel-counterbalanced trunnion structures, it is obligatory so to dispose the counterweighting that the center of gravity of the entire moving mass shall lie upon the axis of rotation; and this generally necessitates the location of a considerable portion of the counterweight above the deck, to the detriment of the appearance of the bridge. Under such conditions it is necessary to provide a heel-balanced, trunnion structure, in which the coincidence of the center of gravity and the axis of rotation are not obligatory, and by which the employment of either pits or unsightly towers is avoided.

These desiderata can be accomplished by a partial balance of span-weight, completing the said balance by means of a counterweight supported on a beam pivotally connected at one end to the heel, and supported at its other end by a roller or truck which passes forward and backward on a track when the span opens or closes, according to a series of patents lately taken out by Mr. Thomas Ellis Brown, Jr.

Either of the two primary types of bascule may be divided into three general classes, viz., trunnion, rolling-lift, and roller-bearing. All of these are good, but none is best for all conditions, nor can it be said absolutely that one is always more economical than another. Each has its good points and each its bad ones; and some are fitted for one location and not for another.

The rolling-lift is sometimes the cheapest, as has been shown often by competitive bids on different types submitted by Contractors; but it is not good practice to adopt it when the pier foundations are of piling, on account of the shifting of the center of gravity of the load on the piles as the span rolls backward and forward, and because of the possibility of pier settlement. The extension and compression of the outer piles, caused by such shifting, has a tendency to crack the superimposed masonry. The great advantage of this type is its retreating bodily out of the way of passing vessels.

The Trunnion type is comparatively simple, but has not the advantage of the retreating span possessed by the rolling lift, and hence often necessitates a rather long leaf for a fixed clear opening.

By the adoption of the Waddell and Harrington detail for bearings, the ambiguity of stress distribution and the secondary stresses, involved through the bending of the ordinary trunnion-axle or trunnion-girder, are entirely avoided, thus rendering the design of structure decidedly more scientific and cutting out some abnormally high intensities of working stresses.

The roller-bearing type has not been much used. When properly designed, it is neat, scientific, and in every essential way excellent, but is not pre-eminently economic.

There is considerable rivalry between the patentees of the various standard types of bascule. Each one seems convinced not only of the superiority of his own type but also of its greater economy; consequently it is not an easy matter to draw conclusions on bascule economics that will satisfy all concerned. This much, however, can be said—when the bascule is entirely a deck structure, the most economic type will depend greatly upon the governing conditions; but when the counterweights, towers, machinery, etc., are above the deck, the Strauss and the Brown types appear to have an advantage over all the others. Until a short time ago the Strauss heel-trunnion type held the record for economy, but the lately-developed Brown Balance-Beam type appears to be slightly more economical than any other bascule in the overhead-counter-weight class. At present there is no example of this new bascule in existence, and the only one yet designed in detail is that for a proposed crossing of the Mystic River at Mystic, Conn. This was designed by the firm of Thos. E. Brown and Son, Consulting Engineers. Estimates of quantities of materials made from their finished drawings indicate that the Browns have succeeded in producing the most economic bascule with overhead counterweight yet evolved.

Economics of Vertical-Lift Spans

The governing conditions which prove economic for the vertical lift, in comparison with the other types of movable span, are as follows:

First. Low vertical clearance.

Second. Large horizontal clearance.

Third. Heavy moving span.

Fourth. Existence of fairly-long flanking-spans.

Fifth. Deep foundations, especially when the flanking-spans are long.

Sixth. Expensive piers, when flanking-spans are long.

Seventh. Skewed crossings.

Eighth. Concrete deck desired.

Ninth. Other first-class deck, especially if heavy.

Tenth. Shifting channel.

Eleventh. High wind pressures to be provided for.

Twelfth. Wide deck.

Thirteenth. Necessity for quick operation.

A low vertical clearance is evidently favorable to the vertical lift. The real factor in this case is the required vertical movement of the lift span. A greater clearance above the water when the span is down favors the vertical lift; since, for any required clear height with the span raised, the vertical movement is reduced.

A large horizontal clearance favors the vertical lift in comparison with the bascule. For a given weight of moving span, the towers, counterweights, and machinery of a vertical-lift bridge are independent of the span-length, while those items for a bascule vary nearly directly therewith.

As will be explained fully later on in this chapter, the ratio of vertical and horizontal clearances for equal costs of bascules and vertical lifts is generally about unity, being somewhat less for short and light spans, and materially greater for long and heavy ones.

Increased weight of span is favorable to the vertical lift. This is chiefly due to the weight of the rear legs and bracing of the towers, which, for a given height thereof, are nearly as heavy for light spans as for heavy ones. For a very light span and high vertical clearance, the weight of the towers may nearly equal that of the span; whereas, for a heavy span and the same vertical clearance, it may be only one-third of the said weight. There is no such variation in the case of the bascule, since the weight of the bracing is a smaller proportion of the total weight of the towers and counterweight trusses.

A layout in which the economic length of the flanking-spans is much greater than the proper length of a bascule tower-span favors the vertical lift. In such a case the rear legs of the vertical-lift towers rest on the flanking-spans without producing any material stresses therein. But in bascules with overhead counterweights it will be necessary to put in an additional pier, or to carry the weight of the counterweight on one of the flanking-spans, or to put the counterweight trunnion over the pier and cantilever the flanking-span out to support the trunnions of the moving span. The first method is most economic where the substructure is cheap, and the third generally where the substructure is expensive. The third scheme requires ample fenders to protect the cantilevered portions from passing vessels. In deep water these fenders may be impracticable or very costly, thus making the second arrangement the best.

Over a canal, or a small canalized river, the layout often calls for a movable span and two short approach spans. In such a case four piers will be required for either the bascule or the vertical lift. This case is nearly always less favorable to the vertical lift than the layout where long flanking-spans are called for.

Deep foundations and expensive piers are favorable to the vertical lift,

as compared with the bascule, when long flanking-spans are employed; but for crossings over canals or canalized rivers the cost of the substructure usually has little effect on the comparison. A crossing where the piers rest on piles or sand is especially favorable to the vertical lift, since the total loads for that type are less than those for the bascule. Rolling-lift bascules are not well adapted to such foundations. Deep foundations are usually unfavorable to the swing, on account of the large base of the pivot pier.

Advantage can be taken of a badly-skewed crossing by the vertical lift; for both the span and the towers may be skewed with very little extra expense, while at least one end of the bascule will have to be squared, thus lengthening the span and increasing the cost. It is true that a similar advantage can be taken with the swing by making both ends skewed, but that would prevent the reversing of ends. However, there is not often any real necessity for such reversal.

A channel that has a tendency to shift will give the vertical lift a great advantage over either of the competing types, because it is the only one of the three which permits a change in the location of the opening span without necessitating excessive expense.

A requirement for high wind pressure militates greatly against the bascule, because it involves an augmenting of the power and, consequently, also the cost of the operating machinery; but it affects hardly at all the cost of either the vertical lift or the swing.

In a modern, first-class structure, where a concrete deck should be used to eliminate fire risk, the vertical lift and the swing can be employed; but the bascule cannot, if there is to be paving on the concrete. While a concrete deck without other paving could be used on a bascule, the span would be so heavy that a vertical-lift bridge would always be the cheaper, except for a very high lift with a short span.

The use of a block pavement or other heavy type of deck favors a vertical lift as compared with the bascule, since, for the latter, extra expense is required properly to fasten the blocks so that they shall not fall off. Furthermore, the greater weight of the deck favors the vertical lift.

The widening of the deck lengthens the moving span in any swing, and increases the size of the pivot pier and the cost of the pier protection. In a skewed crossing it augments the length of a bascule. Since a wider deck involves a heavier structure, this factor also favors the vertical lift.

In respect to quickness of operation, this condition does not affect materially the comparative economics of vertical lifts and single-leaf bascules; but both the double-leaf bascule and the swing are at a disadvantage, since they take fully twice as long to operate as do the other types.

The question of flanking spans is of such importance that the author has found it necessary in his practice and in his economic studies to divide the vertical-lift bridge into two distinct types—one where there are fixed spans flanking the movable span, and the other where there are not.

With regard to the towers, the vertical lifts may be divided into three classes, viz.,

A. Structures with towers having inclined rear legs.

B. Structures with towers having vertical rear legs.

C. Structures having towers, each composed of a single bent, and generally, but not necessarily, connected at their tops by a span or strut crossing the opening.

In respect to the economics of these three classes, it may be stated that, for a combination of a short span and a moderate vertical clearance, Class C is the most economic; but it is not compatible with rigid construction for high clearances—also that Class A is always more economic than Class B, because the latter involves the doubling of the number of sheaves and a considerable increase in the weight of the towers, as well as a small extra amount of wire rope. It is found advantageous, however, in the case of very-badly-skewed structures, because it throws the large and clumsy counterweights entirely outside of the towers and permits of a thorough system of internal sway bracing for the latter.

Economics in Detailing of Vertical Lifts

There are a few economic problems that arise in the detailing of vertical-lift bridges, the principal of which are the kinds of materials for counterweights, the use or non-use of counterbalancing chains, the employment or omission of buffers and the best type of same, the character of pavement base, the location of the machinery house, and the determination of the number and the size of the supporting ropes which will make the combined cost of ropes and sheaves a minimum.

In respect to the cheapest material for counterweights, in most cases it is ordinary concrete; but sometimes, when the space is limited, it pays to make the mass heavier by incorporating in it materials of greater density than that of ordinary stone, such as iron ore or pig iron. The latter was employed entirely for the counterweights of the Halsted-Street Lift-Bridge, the first structure of the type on a large scale ever built; but its utilization was not economic, consequently in subsequent structures its employment was abandoned.

As to whether it is advisable to use chains for the purpose of keeping the main cables always counterbalanced, that is an economic problem which is dependent upon the kind of power used, how often the bridge is to be operated, and the extent of the span movement. They should not be employed for low lifts, as in these the unbalanced rope-load is just about right to hold down the span properly. The author favors the adoption of such chains for most cases where the vertical movement is large, so as to make the peak load of power a minimum and thus keep down the cost of both installation and operation; but he recognizes that, when the price of power is low and the bridge is not to be opened often, it

it would be economical to omit the said chains. These can be made of cast-iron links bored for small pins, so as to keep their pound price as low as possible.

It may be all right to omit the buffers entirely and trust to the automatic brakes to stop the span, but the author prefers to adopt the additional precaution of using the buffers as a safeguard, in case that anything should go wrong with the operation of the brakes. He has tried two kinds of buffers, viz., oil and air, and prefers the latter on account of greater reliability and cleanliness.

In respect to the character of base for pavement, it will generally be economical to use the lightest practicable, consistent with proper requirements for strength and stiffness. The modern, stiffened-buckle-plate floor with a thin layer of concrete thereon supporting three-inch wooden blocks, described in Chapter XXI, will save the lifting of considerable weight; and the consequent reduction in cost of ropes, sheaves, counterweights, and capitalized power will generally more than offset the extra cost of the lighter base.

From motives of economy alone, it is better to place the machinery house in one of the towers instead of on the span; because it takes extra truss-metal to carry it in the latter place, and this extra metal and the weight of the house with its machinery augment the cost of ropes, sheaves, counterweights, and power. But generally the operator obtains a much better view of passing vessels from the middle of the span than he could from the tower or anywhere else, consequently it will then be better to put the house on the span in spite of the extra expense involved by so doing.

Finally, in respect to the determination of number and sizes of supporting cables, it may be stated that the greater the number of cables the smaller their diameter and the smaller the legitimate diameter of the sheaves; but the greater the number of ropes the wider the said sheaves. Again, multiplicity of ropes means a multiplicity of expensive details for their connection; hence the determination of main-rope diameter is a question that generally has to be solved by good engineering judgment based upon experience rather than by economics pure and simple. The author's usual practice is to adopt four ropes per corner for loads up to 250,000 lbs., eight from 250,000 to 1,250,000 lbs., and sixteen for greater loads.

There is a combination of vertical lift and cantilevers that has lately proved to be economic. The author evolved it many years ago, but did not publish anything concerning it, preferring to await the psychological moment for utilizing it. The opportunity did not present itself until 1918, when, as a member of the Board of Advisory Engineers to the Public Belt Railroad Commission of New Orleans, he prepared a low-level-bridge layout with a vertical-lift span for a proposed crossing of the Mississippi River near that city. He had adopted for the movable span a clear opening of three hundred feet; and, when trying to obtain an informal

approval of that length by the then Chief of Engineers of the United States Army, he was told that, in view of the possibility of the Mississippi being navigated in the future by large flotillas of barges, the suggested opening would be too small. Thereupon he made another layout having the same length of vertical-lift span but a clear width between piers of five hundred feet, supporting the towers on the cantilevered ends of the two flanking-spans. This arrangement would permit of the flotilla steamer passing through the opening beneath the raised lift-span and of the barges slipping under the cantilever arms, in case that the current should swing the flotilla broadside to the structure, the said cantilever arms having sufficient vertical clearance above extreme high water to permit such passage under any river condition. When the author submitted this new layout to the local U. S. Engineer officer in charge at New Orleans, the request was made for an economic investigation of the structure with not only the suggested clear openings of 300 and 500 feet, but also with those of 600 and 700 feet, adopting a lift span of 350 feet for the 600-ft. opening and a 400-ft. one for the 700-ft. opening. The result of the investigation showed that the 500-ft. opening was more economic than the 300-ft. one, that the latter made the total cost of structure about the same as did the 600-ft. opening, but that the 700-ft. opening was so decidedly uneconomic as practically to be prohibitive.

Early in 1920 the author's Indian agents in Calcutta wrote asking him whether he could evolve a design for crossing the Hoogly River at their city by a single span in a manner that would comply satisfactorily with certain unusual and extremely drastic physical conditions. These conditions were met by the utilization of the above-mentioned scheme of a vertical lift and cantilever arms, combined with pier foundations of built-up piles of exceedingly great length sunk by jetting. This type of piling was originated by the author so many years ago that he had actually forgotten about the matter and had to resurrect the drawings from an ancient office-file. The layout suggested is shown in Fig. 30*a*. As can be seen by inspection, it contains still another economic innovation, viz., the supporting of the counterweights, which balance the weight of the moving span, from the tops of the columns over the main piers, thus relieving both the cantilever arms and the anchor arms from stresses due to the said counterweights. There were some other economic innovations involved in the study and estimate that are not shown on the layout; but it is not necessary to discuss them here.

An alternative design was submitted at the same time to the agents mentioned by substituting a double-leaf bascule for the vertical-lift span, with the statement, however, that the first-described layout is in every way preferable, excepting only for the fact that the bascule design gives an unlimited vertical clearance. The Brown wire-rope type of bascule was used; and the counterweights were placed over the main piers, as in the vertical-lift design.

Unfortunately, the "Powers" at Calcutta have not yet seen fit to give these layouts consideration; and it seems probable that the time and gray matter expended by the author in evolving this solution of a knotty and interesting problem will be wasted—at least from a pecuniary point of view—hence, in order that such waste may not be total and permanent, he is now presenting to the engineering profession the results of this economic study.

Comparative Costs of Various Types of Movable Spans

The collection of the necessary data concerning the quantities of materials in movable spans has been no easy task, because the designers and builders of such structures seldom publish the total weights of metal involved, nor, what is equally important, the division of the said weights into various logical groups, such as moving span, towers, counterweight trusses, etc. Furthermore, such records as can be collected need to be carefully plotted and compared on some logical basis, since different bridges are designed for various specifications and often under dissimilar conditions.

The analyses of all these weights have been prepared for this economic investigation by the author's assistant engineer, Mr. Shortridge Hardesty. Comparisons have been made with great thoroughness between vertical-lift bridges and both the heel-trunnion and the Brown balance-beam, single-leaf bascules; firstly, because it was possible to secure the fullest data concerning these types; secondly, because, for many layouts, the logical choice would be one of them; and, thirdly, because it is a comparatively simple matter to contrast them fairly and definitely. Swing spans and deck bascules with underneath counterweights have also been investigated, but questions of types of piers, distance from grade to water line, æsthetic considerations, and other factors affect the comparisons so largely as to make the results considerably influenced by the personal equation of the designer and by the conditions of each individual case. Railway bridges have been used as a basis for the most part, because the best records available deal with that class of structure, and because there is less variation in them than in highway bridges.

For the vertical-lift bridge the author had at hand the records of some thirty cases designed in his office, supplemented by complete curves of weights of towers for different heights thereof and for various weights of moving spans. The weights of the different machinery groups, such as ropes, sheaves, equalizers, operating machinery, etc., were plotted in terms of the weight of the moving span and the height of lift. Curves of average weights were then drawn for each group. The same was done for each of the items of structural metal. These various curves, after being first drawn in terms of the weight of the moving span, were replotted in terms of the weight of a fixed span of the same length and carrying capacity.

vertical-lift span with the machinery and motors in a house at the center of the span weighs over ten per cent more than the corresponding fixed span.

For the railway, heel-trunnion bascule there were available complete detailed weight-records of seven bridges, complete detailed estimates for several more prepared by their designers, and summarized estimates for about a dozen others. Most of them were for double-track-railway bridges. The percentages of the weights of the towers (less the tower floor-systems), the counterweight trusses and girders, the links, the operating struts, etc., in terms of the weights of the moving span were then figured, the machinery girders being included with these items regardless of whether the motors were on the span or in the tower. The percentages were then computed in terms of the weight of the corresponding fixed span, the bascule leaf being a few per cent heavier than the said span. These latter percentages were then plotted with the lengths of moving spans as abscissæ. These plots provided a sufficient number of points for the drawing of fair average curves for double-track bridges. The weights of trunnions, pins, and machinery, in percentages of the weights of the corresponding fixed spans, were then plotted, and an average curve was drawn.

Fig. 30*b* gives the resulting curves for both bascules and vertical lifts. The full lines for the tower and counterweight steel of the vertical lift apply when there are flanking truss spans, and the dotted lines when there are no flanking truss spans. The crosses on the bascule plots indicate bridges for which there were complete weight-records, and the circles refer to structures for which there were full, detailed estimates.

These plots give fair, average curves for the quantities in both the vertical lift and the bascule, all in percentages of the weight of a simple span of the same length and carrying capacity. This basis of comparison was adopted because it eliminates the effect on the moving span of different live loads, different specifications, and different weights of decks. Also, the percentages derived in this manner can be applied with fair accuracy to highway bridges. These curves made it a comparatively simple matter to contrast the costs of the superstructures of the two types.

The curves of Fig. 30*b* are drawn for well-designed bridges, and do not represent the lightest structures of these types that it is possible to build. The factors of safety of the wire ropes of the vertical-lift bridges have been taken somewhat larger than the author, from his own experience, considers necessary, in order to meet somewhat the desires of railway bridge engineers.

For swing spans the data given in Chapter LV of "Bridge Engineering," supplemented by other data in the author's office, proved ample.

There were also available the complete quantities for the double-leaf trunnion-bascule recently designed by the author's firm for the highway bridge over the Housatonic River, and those for the single-leaf, Brown-balance-beam-bascule highway-bridge over the Mystic River. A vertical-lift bridge was estimated for each location, and the results compared.

When contrasting the vertical lifts and the heel-trunnion bascules, it was assumed that, for low vertical clearances, the lengths of moving spans would be the same in the two types, the fenders being placed as close to the piers as possible. Since the bascule can rarely be rotated through more than 83 or 84 degrees, a vertical line through the face of the fender

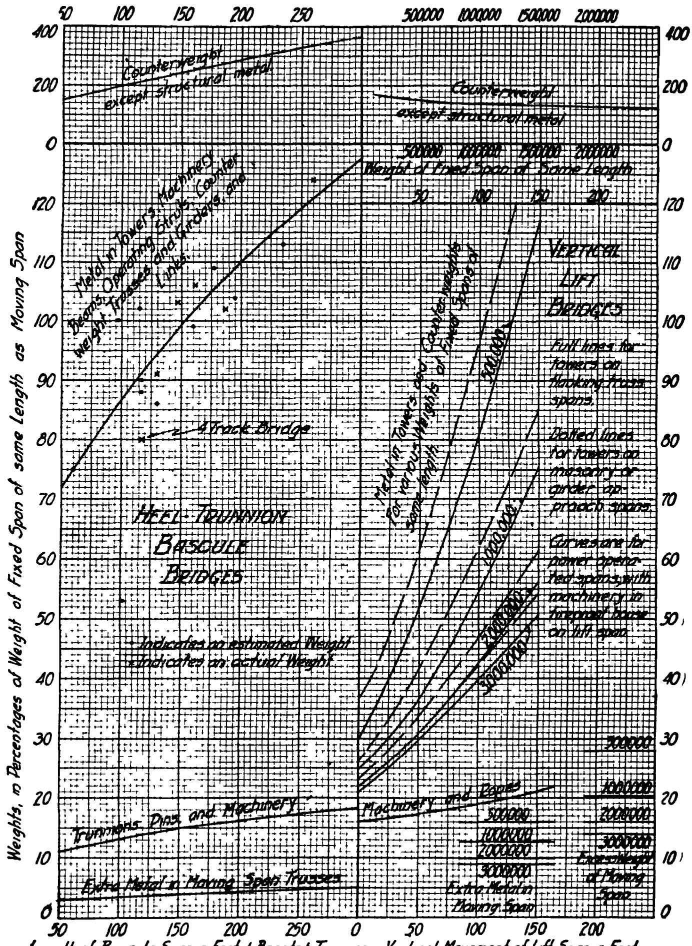
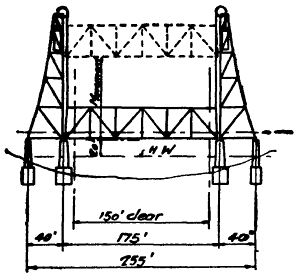


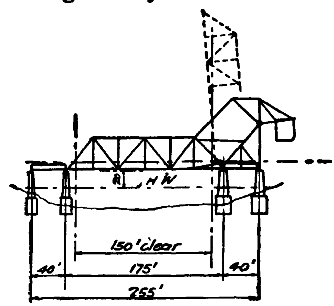
FIG. 30*b*. Percentage Weights for Double-Track-Railway, Vertical-Lift Bridges and Single-Leaf, Heel-Trunnion Bascules.

near the trunnion will generally intersect the bottom chord of the fully-raised span at some height above the water. This height will depend upon the horizontal distance from the trunnion to the said vertical line, the height of the trunnion above the bottom chord, and the angle of inclination of the said chord to the vertical; but it will generally be about 50 or 60

feet. For clear heights much exceeding these, the bascule leaf will have to be longer than the vertical-lift span, unless an encroachment on the clearance at one corner be permitted, which is not usually the case. For a clear height of 150 feet, this excess length will generally be at least 10 or

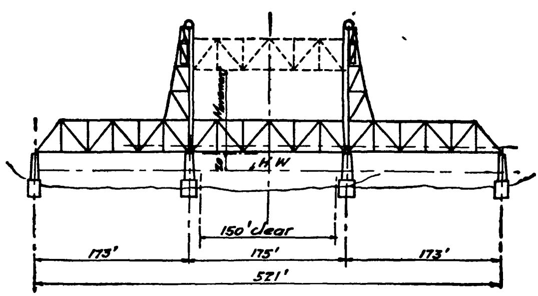


Vertical-Lift Bridge

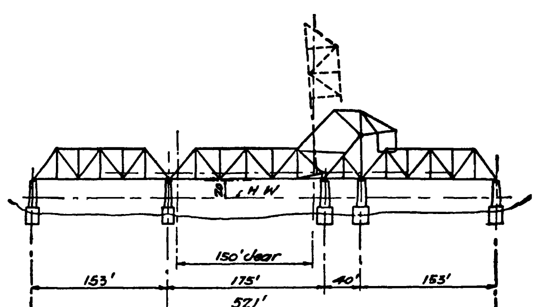


Heel-Trunnion Bascule

FIG. 30*c*. Layouts with No Flanking Truss-Spans.



Vertical-Lift Bridge



Heel-Trunnion Bascule

FIG. 30*d*. Layouts with Flanking Truss-Spans.

12 feet, and often still more. This limitation does not apply to the Brown balance-beam type, which can be rotated 90°.

In making the comparisons, it was necessary to consider two cases—first, when there are long flanking-spans, and, second, when there are not. Typical layouts are shown in Figs. 30*c* and 30*d*. It will be noted that in

each comparison the total length of bridge considered is the same. The substructure was designed for several cases—piers on rock at various depths, piers on deep-sand foundations, and piers on piles loaded to the limit of 30 tons each. Calculations were made for clear-channel widths of 100 feet, 150 feet, 200 feet, and 250 feet.

For the layouts shown on Fig. 30*c*, in which there is the same number of piers in the two cases, it was found that ordinarily the substructure costs nearly the same for the two types. In some instances the vertical-lift substructure was cheaper, while in others that of the bascule was a trifle more economic. The comparison for these layouts, consequently, is almost entirely a question of superstructure costs. For the layouts shown in Fig. 30*d*, the bascule substructure is always the more expensive, so that both substructure and superstructure must be considered.

Fig. 30*e* gives comparative costs for double-track-railway-bridges designed for Class 60 loading. The following unit prices were used:

Structural metal in spans	8¢ per lb.
Structural metal in towers, counterweight trusses, etc.	10¢ "
Machinery of all kinds	40¢ "
Counterweights	$30 per cu. yd.
Pier shafts	$20 " "
Pier bases	$40 to $60 " "

The lower group of curves, for layouts such as shown in Fig. 30*c*, gives superstructure costs only; while the upper group, for layouts such as indicated in Fig. 30*d*, records the total cost of superstructure and substructure, with the piers resting on piles. The comparison for the latter substructure condition gives average results, and was therefore adopted. For other types of substructure the relative costs differ but slightly, excepting that the bascule is considerably more costly with deep or expensive foundations. The full lines for the bascule costs, noted "Clear Height 50 ft. or less," apply for greater heights when an encroachment on the corner of the clearance diagram is permitted.

Fig. 30*f* gives, for various clear-channel widths, the vertical clearances at which the vertical-lift bridge will just equal the bascule in cost. This is shown for layouts both with and without long flanking-spans. The two full lines apply for clear heights up to 50 feet, and for greater heights when an encroachment on the corner of the clearance diagram is allowable; while the two dotted lines are for greater heights with no encroachment permitted.

In working up the curves of Figs. 30*d* and 30*e*, it was assumed that the distance from center to center of piers exceeds the clear channel by from 20 to 30 feet, and that the clearance above the water line is 20 feet when the moving span is down. With a smaller down-clearance than this, the clear heights at which the types are equal will be reduced; and with a greater down-clearance it will be increased.

The curves of Figs. 30*e* and 30*f* are based upon the assumption that, for the vertical lift span, the motors, operating machinery, and machinery house are located on the moving span, thus increasing the weight of the said span by 10 per cent or more. While this is the ideal location for this machinery, especially when the operator is on the span, it is considerably

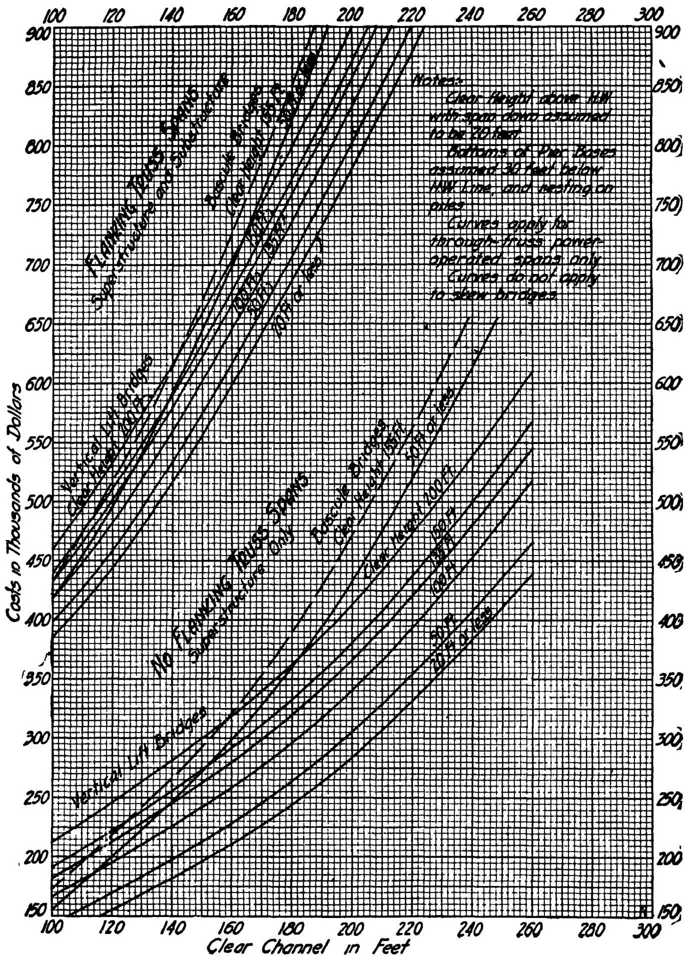


FIG. 30*e*. Comparative Costs of Double-Track-Railway, Vertical-Lift Bridges and Single-Leaf, Heel-Trunnion Bascules.

cheaper to place the said machinery in the towers. If this be done, the clear heights at which the cost of the vertical lift will just equal that of the bascule will be increased 10 feet or more; but the author has plotted the curves on the other basis, because he considers the additional cost justified.

The curves do not apply to skew layouts. If in these the piers are to be square, Fig. 30*f* can be used by making a layout and finding the required distance from center to center of piers, subtracting therefrom a length varying from 20 feet for a 100-foot span to 30 feet for a 200-foot span, and entering the diagram with the result as the "clear-channel" width. If

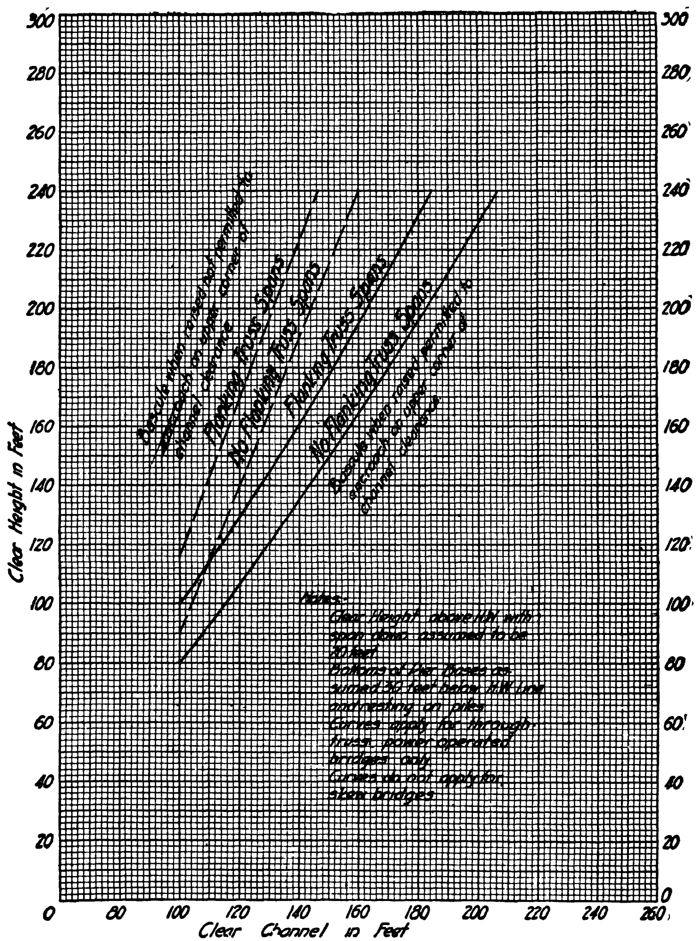
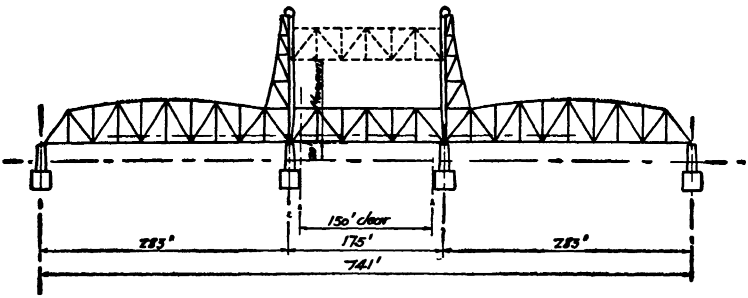


FIG. 30*f*. Clear Channels and Clear Heights for Equal Costs of Double-Track-Railway, Vertical-Lift Bridges and Single-Leaf, Heel-Trunnion Bascules.

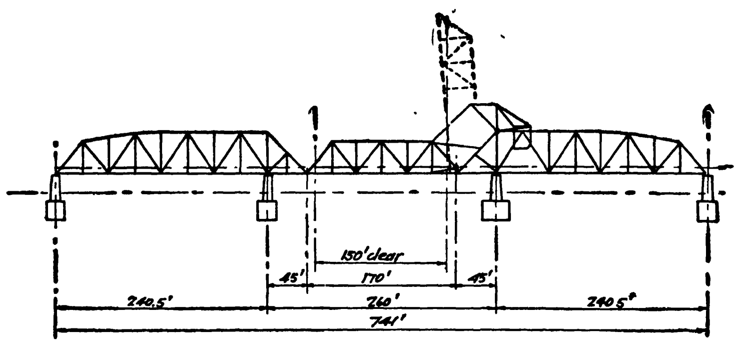
skewing piers is permissible, both can be skewed for the vertical lift, but only the rest-pier for the bascule. This condition favors the vertical lift materially.

For deep and expensive piers, a bascule layout, such as is shown in Fig. 30*d* is not economic, that indicated in Fig. 30*g* being cheaper. The

two layouts shown in the latter figure were next compared. It was found that, whereas, for the layouts of Fig. 30*d*, the bascule and the vertical lift were of equal cost for a clear height of about 170 feet, for the layouts of Fig. 30*g* they would be equal for a clear height of 125 feet with



Vertical-Lift Bridge



Heel-Trunnion Bascule

FIG. 30*g*. Layouts with Flanking Truss-Spans—Bascule Tower Cantilevered.

rock foundations, and for one of 180 feet with deep-sand or pile foundations.

In order to illustrate the use of the curves of Figs. 30*b*, 30*e*, and 30*f*, the following problems and their solutions are given:

Example No. 1

What are the quantities of superstructure materials in a bascule and in the corresponding vertical-lift span for a Class 60, double-track-railway bridge, the span-lengths being those in Fig. 30*c*, the clearances above high water being 20 feet with span down and 150 feet with span raised to full height, and the bascule span being permitted to encroach in one corner of the waterway clearance?

Distance center to center piers.......................... = 175 feet
Distance center to center of bearings of movable span.... = 170 feet
Vertical movement of lift span. (= 150–20) = 130 feet
Weight of 170-ft. fixed span.

Deck	900×175 =	158,000 lbs.
Metal	4,630×170 =	787,000 lbs.
Total...........		945,000 lbs.

Vertical-Lift Bridge:

Structural metal.

Lift span...............	787,000+945,000×0.13	=	910,000 lbs.
Tower floor-systems......	2×1,320×40	=	105,000 lbs.
Total span metal........................			1,015,000 lbs.
Towers and counterweight steel.................	945,000×0.80	=	760,000 lbs.
Total................................			1,775,000 lbs.
Counterweights.........	$\frac{945,000\times1.20}{3800}$	=	300 cu. yds.
Machinery and ropes.....	945,000×0.20	=	190,000 lbs.

Bascule:

Structural metal:

Bascule span............	787,000+945,000×0.04	=	825,000 lbs.
Tower floor-system and approach span........	2×1320×40	=	105,000 lbs.
Total span metal.......................		=	930,000 lbs.
Towers, counterweight trusses, etc.	945,000×1.04	=	980,000 lbs.
Total...............................		=	1,910,000 lbs.
Counterweights..........	$\frac{945,000\times2.60}{3800}$	=	640 cu. yds.
Machinery and trunnions.	945,000×0.15	=	142,000 lbs.

It will be noted from the foregoing that the estimator must be able to figure independently the weights of the 170-foot fixed span, the tower floor-systems, and the approach spans. The curves of Chapter LV of "Bridge Engineering" can be used for this purpose.

Example No. 2

In the preceding example, what would be the comparative costs of the two superstructures, using the unit prices for materials in place which were employed in working up Fig. 30*e*.

Vertical Lift:			
Deck	510 lin. ft.	@ $10	$5,100
Metal in spans	1,015,000 lbs.	@ 8¢	81,200
Metal in towers, etc	760,000 lbs.	@ 10¢	76,000
Counterweights	300 cu. yds.	@ $30	9,000
Machinery and ropes	190,000 lbs.	@ 40¢	76,000
Elec. equip. and houses			20,000
Total			$267,300
Bascule:			
Deck	510 lin. ft.	@ $10	5,100
Metal in spans	930,000 lbs.	@ 8¢	74,400
Metal in towers, etc.	980,000 lbs.	@ 10¢	98,000
Counterweights	640 cu. yds.	@ $30	19,200
Machinery and trunnions	142,000 lbs.	@ 40¢	56,800
Elec. equip. and houses			20,000
Total			$273,500

Entering the lower group of curves of Fig. 30*e* with a 150-foot-clear height for the vertical lift, we find the cost to be $270,000; while entering it with a 150-foot channel and a clear height of "50 feet or less" for the bascule (because encroachment on one corner of the waterway clearance is permitted), we read $272,000 as the cost of the bascule. The margin in favor of the vertical lift is, therefore, given as $2,000 by Fig. 30*e*, and as $6,200 by the figures derived from the more accurate curves of Fig. 30*b*, which is a satisfactory check.

Example No. 3

What are the comparative costs of a double-track-railway, Class 60, bascule and the corresponding vertical lift for a 180-foot clear-opening flanked by fixed truss spans, the clear height required being 135 feet when the span is raised and 15 feet when the span is down, the bascule being permitted to encroach on one corner of the waterway clearance.

For the vertical-lift span, we must enter the upper group of curves in Fig. 30*e* with a "clear height" of $135-15+20=140$ feet; for that diagram is plotted for a clear height of 20 feet with span down. For a 180-foot

channel, we find the cost to be $770,000. For the bascule, we enter with a "clear height" of "50 feet or less," and find the cost to be $820,000.

Example No 4

What would be the comparative costs in Example No. 3, if the bascule were not permitted to encroach on one corner of the waterway clearance?

The cost for the vertical lift is $770,000 as before. Entering Fig. 30*e* for the bascule with a "clear height" of 140 feet, by interpolating we find the cost to be about $845,000.

Example No. 5

What are the comparative costs of a double-track-railway, Class 60, bascule and the corresponding vertical lift for a 120-foot clear-opening without flanking-spans, the "clear height" being 50 feet with span raised, and 20 feet with span down?

From the lower group of curves in Fig. 30*e*, we find the cost of the vertical lift to be $170,000, and that of the bascule $200,000.

Example No. 6

What would be the comparing costs for Example No. 5, if flanking truss spans were used?

Entering the upper group of curves of Fig. 30*e*, we find $460,000 for the vertical lift, and $497,000 for the bascule.

Example No. 7

In a double-track-railway bridge, with flanking truss spans, having a clearance above high water, with span down, of 20 feet, what will be the vertical clearances for a vertical-lift span of equal cost with a bascule, when the clear horizontal opening is 100 ft., 110 ft., 120 ft., or 130 ft.?

From Fig. 30*f* we find the following:

Horizontal Clearance	Vertical Clearance	
	Bascule Permitted to Encroach on Corner of Clearance	Bascule not Permitted to Encroach on Corner of Clearance
100′	100′	116′
110′	115′	143′
120′	129′	170′
130′	145′	197′

Example No. 8

What would be the results in Example No. 7 in case there were no flanking truss spans?

From Fig. 30*f* we have the following:

Horizontal Clearance	Vertical Clearance	
	Bascule Permitted to Encroach on Corner of Clearance	Bascule not Permitted to Encroach on Corner of Clearance
100′	80′	90′
110′	93′	115′
120′	106′	140′
130′	120′	165′

Comparisons were made for swing bridges giving two 150-foot channels and two 200-foot channels, as against one channel of 150 and one of 200 feet for the vertical lift and the bascule. It was found that the swing was a trifle more expensive than the bascule for the 150-foot channel, and of about the same cost for the 200-foot channel. For clear heights less than 160 feet, the vertical lift was cheaper than the swing. There was some variation with the depth of the foundations, the swing being more expensive for deep ones.

The Mystic River, Brown-Balance-Beam bascule is a through, plate-girder, highway bridge 215′ 3″ long, consisting of a fixed span of 68′ 6″, a tower span of 23′ 9″, a bascule span of 88′ 0″, and a fixed span of 35′ 0″. The clear channel is 75′ 0″. This layout was compared with one for a vertical lift, consisting of one 92′ 3″ fixed span, one 88′ 0″ lift span, and one 35′ 0″ fixed span. It was found that the two were of equal cost when the required vertical movement of the lift span was 61′, corresponding to a clear height of 64′, since there is only a three-foot clearance when the span is down. If the clearance with span down had been the usual one of 15 or 20 feet, the two types would have been equal for a clear height of 75 or 80 feet. This Brown bascule rotates through 90°, so that the moving span never needs to be any longer than that of the vertical lift.

The Housatonic River Bridge is a concrete-arch structure, with a simple-trunnion, double-leaf-bascule span giving a clear waterway of 125′, the distance from center to center of trunnions being 175′. The bascule piers were necessarily quite heavy and massive; and while much lighter ones would have sufficed for the vertical lift, it was decided to make a compari-

son with the same sized shafts, reducing the pile bases for the smaller loads of the vertical lift. This was the most favorable assumption for the bascule. It was found that the two types were of equal cost for a clear height of 180 feet, corresponding in this case to a vertical movement of 155 feet for the lift span.

It has long been the author's surmise that where the clear height required does not exceed the clear width of channel, the vertical lift would always be cheaper than the bascule. The curves of Fig. 30*f* show this to be true when the bascule is not permitted to encroach on one corner of the clearance. For cases where such encroachment is permissible, the statement is always true for bridges with flanking-spans, and practically so for bridges without flanking-spans.

It would be very valuable to extend this investigation to cover short spans, where the bascule would usually be of the trunnion type with either the underneath or the overhead counterweight, or of the rolling-lift type; while the vertical lift would have two-leg towers with over-head-bracing trusses. However, an investigation of this sort would require more data than those at the author's disposal. From comparisons made in the past, it is evident that, *for such short spans*, with the standard low clearances of the inland waters the vertical lift will almost always be the cheaper, while with the high clearances required along the coast one of the bascule types will be more economic.

CHAPTER XXXI

ECONOMICS OF OPERATING MACHINERY AND POWER

THE data for this chapter were furnished mainly by the author's old friend and occasional associate in professional work, Thomas Ellis Brown, Mem. Am. Soc. C. E.; but some valuable suggestions as to items to be considered were given by Major Leon L. Clarke, who, for many years before the Great War and for a short time after his return from France, where he rendered effective and distinguished service to the Allied Cause, was the author's principal assistant mechanical engineer, and as such devoted his entire attention to the designing and installation of machinery for operating movable spans.

As long ago as 1892, when the author was retained by the City of Duluth, Minn., on his first design for a vertical-lift bridge, he recognized the necessity for some expert aid in solving certain important mechanical problems; and, consequently, he looked the country over, in order to find the highest American authority on the mechanics of lifting great weights. The result of his search proved that, even at such an early date, Mr. Brown was universally acknowledged to be the best authority on elevators and their machinery; and, therefore, the author retained him. From that incident there resulted a friendship and a somewhat desultory association which have proved very satisfactory and beneficial to both parties thereto.

With the help of two such experts as Mr. Brown and Major Clarke the author feels that he has done his best to treat one of the most difficult branches of engineering economics.

The kind of power and type of machinery suitable for the operation of movable bridges depend greatly upon the nature of the design of the structure, the available space for apparatus, and the local conditions of fuel or power supply, and are, therefore, contingent upon the ruling features of the particular case; hence no hard-and-fast rules of economics therefor can be formulated. On that account the contents of this chapter will be limited to a dissertation concerning general conditions and the offering of a few pertinent suggestions.

The selection of motive power is largely dependent on the location of the bridge with relation to sources of power-supply. When located in or near a large city or town, electric current, either direct or alternating, is almost

always available; and, when purchasable at reasonable meter-rate, it is usually the most convenient and economic type of energy to adopt. Formerly, electricity not being available, steam was invariably used, and boilers were installed on or near the bridge. This involved the handling of coal, disposition of ashes, and continuous maintenance of steam pressure; and it was usually found that the fuel expense was nearly constant and almost entirely independent of the number of movements of the bridge. The author knows of bridges where steam is maintained during long periods of inactivity, and during periods of practically closed navigation, thus rendering the operation excessively uneconomic. It is evident that a form of power which may be paid for only as usefully expended is of true economic value. In some large cities of Europe hydraulic pressure or compressed air is purchasable by meter; and bridges erected within reach of such pressure systems can most advantageously be operated thereby, as these types of power give a smooth and perfect control attainable in no other way. In the United States, however, we are confined practically to two sources of power other than steam, viz., electric current and the internal combustion engine; and as electricity is quite generally available, it is most commonly used, but few instances existing where internal combustion engines are employed alone as the primary motive power, though quite often as auxiliaries for emergency use.

With the great improvement reached in the design and construction of the high-speed, multi-cylinder engine of the present day, there seems to be no good reason for its limited employment in the operation of bridges. In many cases the cost of transmission lines, transformers, and other paraphernalia necessary to convey electric current to the bridge site, would exceed the cost of such engines, while the fuel for them is obtainable at almost any cross-roads, consequently true economics demands their more extensive use.

Efficiency, as commonly understood, i.e., ratio of power output to power input, rarely needs consideration in the economics of movable bridges, as the time of motion is generally not much over a minute, or at most two minutes, in one direction,—say on an average three minutes per cycle; and comparatively few bridges are called upon to open more than 10 or 20 times per day on an average throughout the year. Thus the total yearly working time of a fairly-active bridge will ordinarily amount to less than 75 actual days of motion, the power consumed being mainly that required for starting and stopping; and, therefore, efficiency as above defined is of but little importance. True economics demands the selection of power and machinery from the standpoint of as great simplicity and low first cost as is consistent with robustness and durability, and especially with positiveness and ease of control.

In the earlier days of electric supply (and this condition may possibly apply in a few localities today) it was customary to charge a flat monthly or yearly rate for current, based on the peak load of the motors used, with-

out regard to the actual amount of current consumed; and, should such a case arise, other forms of power should be carefully considered.

When electricity is purchasable, the consumer is usually confined to the kind of current the producing company will supply, and, therefore, has no choice between alternating and direct current; but whenever such a choice exists, direct current should have the preference, as giving much greater latitude in motor speeds, and, consequently, tending to economy in cost of gearing, and, more important still, as giving a much more flexible and perfect control, especially when shunt-wound fields are used, providing automatic speed-control under positive and negative loadings.

The more usual sizes of movable bridges, requiring motors of from 30 to 100 H.P., when located adjacent to electric plants of reasonable size, may, preferably, be operated directly from the line, if the current be of low tension, or through transformers when it is of high tension; but cases may arise, especially with bridges of very large size, where the power station or the transmission lines cannot stand the starting current or peak load required. In such cases some form of accumulator must be used to extend the current draught over a longer time, so that the motors may be relatively small and within the capacity of the power supply. Electric accumulators (storage batteries) may be used, and the apparatus may be entirely electrical; but such cases, in the author's opinion, offer an excellent opportunity for the use of hydraulic power for the direct movement of the bridge, the primary electric power being employed to produce and store hydraulic pressure in suitable accumulators. This system may be employed also in cases where gasoline engines are desired and where engines of sufficient size to operate the bridge directly would be inconvenient or impracticable; as comparatively small engines could be used to store hydraulic power or compressed air in suitable accumulators.

The conveyance of power from the driving motor or engine to the bridge itself is usually accomplished by gearing, the direct connection to the bridge being by means of gear teeth in the form of strutted racks pivoted to the bridge, or by means of curved segments, forming a gear wheel of large radius bolted directly to the bridge girders; and, in the case of vertical-lift bridges, by means of ropes connected to the towers and wound upon drums carried on the moving span.

In all of these cases, as the movement of the span is slow in comparison with the speed of the motor or engine, long trains of spur gearing are necessary; and the motor or engine should be of as slow speed as practicable in order to reduce the amount of gearing to a minimum.

Thus a bascule bridge, operating through about 90° in one and a half minutes, moves at a speed approximately equivalent to $\frac{1}{6}$ of a revolution per minute; and to gear this to a motor running at, say, 600 R.P.M. means a total speed reduction of 3600:1. For that reason, the primary reduction directly at the moving span should be as great as practicable, i.e., the main

pinion should have the minimum number of teeth consistent with strength and smooth running, and the rack should have as large a radius as the design and dimensions of the bridge will permit. In the case of vertical-lift bridges operated by ropes wound on drums, a considerable saving in amount of spur gearing can be made by running the operating ropes over pulleys, forming a tackle, whereby a ratio of 2, 3, 4, or even more could be obtained by the roping alone.

Much spur gearing in many cases could be eliminated by the use of worm gearing, which, with the perfection of workmanship now attained, is far more efficient than commonly supposed; and, in general, it may be stated that the efficiency of a well-designed and properly-cut worm-gear will be higher than that of its equivalent in spur gearing. It is believed that worm gearing could be introduced advantageously in many cases, in order to reduce the amount of gearing, or, on the other hand, to increase the speed-ratio and enable smaller and higher-speed motors to be employed. Incidentally, the introduction of worm gearing would eliminate the noise of the high-speed, steel spur-gearing quite commonly employed.

The modern multi-cylinder gasoline engine seems ideal for the operation of movable bridges, as it probably combines the greatest power and economy of operation with the least first cost, weight, and space; but it has not been used extensively on bridges on account of its high speed, 800 to 1200 R.P.M. or more, and the consequent large speed-reduction necessitated—also on account of the requirement for friction clutches to enable the engine to be thrown into and out of gear, because such engines must be started before beginning to move the bridge, and must be run continuously during the cycle of operations. These difficulties can be overcome by the use of a hydraulic reducing-gear attached to the engine shaft and in turn driving a worm gear. By such an arrangement the clutches are eliminated, and a speed reduction of from 40 : 1 to 100 : 1 may be obtained at once and the remaining reduction be made with a short train of gears.

The author believes the ideal apparatus would be two high-speed gasoline engines, each with hydraulic transmission gear and worm gear, the two engines together of sufficient power to operate the bridge at the maximum required speed, and one alone (by means of the hydraulic reducing gear) capable of running it at slower speed. A third engine, smaller than the others, should be used to compress air for air brakes, to work end locks and rail locks, and to circulate cooling water for the transmission gears. The third or smaller engine could be arranged to be thrown into gear and used as an emergency engine, in the event of complete break-down of the main engines, so as to operate the bridge at very slow speed; and it could also be utilized for driving a small dynamo for signal and other lighting. Thus all the functions of the bridge could be performed by means of gas engines, and be made entirely independent of outside sources of power supply. The installation and care of transmission lines and transformers would be entirely avoided; and, obviously, the arrangement proposed would be of

the greatest value in situations where electric power is not available, or not obtainable on reasonable terms.

Movable bridges may be considered as consisting of three main elements, the first comprising the actual bridge or carrying structure, fixed in position during normal traffic, and which must be removed to permit of transverse traffic; the second comprising means to support the carrying structure together with the counterweights required to balance it; and the third comprising means to operate or impart motion to the carrying structure, its counterweights, and such parts of the supporting structure as must move therewith—the whole constituting a complete machine.

Of this machine it is convenient to classify those parts which require to be made in the machine shop as "machinery," in contradistinction to those fabricated in the bridge shop and classified as "*structural*"; and, as machine-shop work is much more costly than structural work, economics requires that the design should aim at a preponderance of parts which may be finished in the fabricating shop and at as complete a separation in manufacture between the machinery and structural parts as possible, and also that it should permit of simple and easily-adjustable connections in the field. The vertical-lift bridge meets this requirement much better than does either the swing or the bascule.

The carrying structure will consist essentially of structural steelwork and its adjuncts, such as flooring, tracks, and the like, and will comprise no machinery; the supporting structure will generally consist essentially of structural steelwork carrying such supporting-machinery parts as trunnions, rollers, hangers, sheaves, and the like; while the operating machinery will consist of actuating-or-retarding-machinery parts, such as gears, screws, rams, brakes, buffers, and similar component elements.

The supporting machinery parts are dependent for size and strength purely on the weights carried by them, hence their cost is a function of the weight of the movable masses; and, as they necessarily have slow and limited motions, and as they work under very heavy loads, they are essentially costly and must be designed for the highest permissible unit-bearing-pressures, in order to obtain the smallest practicable diameters. In general, in movable bridges the lengths of bearings are limited by space conditions, and increased bearing surface can only be obtained by enlarged diameters; and, as the weights of bearings, trunnions, etc., increase as the square of their diameters, and their costs nearly in the same proportion, the economic importance of high unit-bearing-pressures will be readily understood, consequently for these parts one is justified in adopting the best grades of bearing metals and the most complete provision for lubrication. The latter involves not only suitable oil-or-grease-grooves, pressure lubricators, and lubricant, all arranged so the said lubricant will surely reach the surfaces under pressure, but also requires convenient and safe means of access, such as steps, ladders, platforms, and railings, as well as

ample light and space to insure the comfort and safety of the attendants, as parts dangerous or even difficult of access will otherwise almost surely be neglected.

The operating machinery, the function of which is not only to transfer the motion of the motors or prime movers to the bridge structure, but also to retard and arrest that of the span, is dependent for the size and strength of its parts on the power or torque of the said prime movers and the resistance of the retarding mechanism or brakes; for the maximum operating stresses to which these parts will be subjected are obviously those which the motive power or brakes can put upon them—and usually those produced by the brakes are the more severe.

Breakdowns of machinery rarely occur from the accelerating force of the motive machine, while many do occur from excessive retardation or brake resistance; and this is easily accounted for, as the maximum torque of the motive machine can readily be determined and cannot be exceeded, while the retarding or brake forces are dependent on frictional resistances, more or less indeterminate, and are liable to great variation with slight changes of condition, rapidity of application, and maladjustment, and also to a very natural tendency on the part of designers and operators to provide unnecessary brake strength. In the author's opinion, the operating brake-resistance should not exceed the starting torque of the motors.

Economics, therefore, requires that the conditions of traffic should be carefully studied, in order to determine as nearly as possible the actual service the bridge must render, as well as the resistances to motion likely to be encountered, with the view of avoiding unnecessarily-great motive-power with correspondingly-larger brake-resistance, and consequent unnecessarily-high first-cost of machinery and motors.

The power and strength of the operating machinery, in the case of swing bridges having equal arms, is mainly a question of speed of operation; as it will generally be found that when the power is sufficient to overcome the frictional resistances and give the acceleration needed to perform the required movement in the required time, there will be an ample amount to overcome any unbalance of wind pressure likely to occur on the arms of the bridge. Similarly with lift bridges, if there is ample power to overcome the normal friction and give the required acceleration, there will be ample power to overcome at full speed any added friction on the guides from side wind-pressure, and, at a slower speed, any unbalanced load on the floor, except, perhaps, in situations liable to unusual snow load.

With the bascule bridge the case is usually different from that of the swing or the lift bridge; because, with the former in a raised position, the floor area exposed to horizontal wind pressure is large, especially in the case of the closed floors of highway bridges. The wind moment about the axis of a bascule bridge is, therefore, great, while the lever arm of the operating machinery is usually comparatively short; and hence the pressures and forces in the operating machinery, as well as the motive power

required (and, consequently, the cost), will generally be determined by the wind pressure to be resisted rather than by time of operation and acceleration. It is thought that the tendency of engineers is arbitrarily to assume unnecessarily-high wind-pressures, without regard to the actual conditions and situations, and that, if more careful studies of the surroundings were made, and if the pressures provided for were more in accordance with those likely to occur in actual practice, considerable economy would result.

The author believes that in but very few situations are bascule bridges liable to be operated under anything like such wind pressures as are often specified, and that due consideration is not given to the fact that, even though the toe of the span when high in the air may be under considerable wind pressure, this pressure does not extend to the heel of the span, and that on the whole surface the total pressure is far less than the velocity of the wind would seem to indicate. This is especially so when, as is often the case, the banks are high, or covered by trees or buildings, and the bridge is sheltered from all but winds in the direction of the channel. The author believes, too, from both his own experience and conferences with other bridge specialists, that, in general, bascule bridges capable of slow operation against a uniform pressure of ten pounds per square foot, or even less, and at full speed against two pounds per square foot will answer all requirements.

It is quite usual with bascule bridges to specify holding or brake power sufficient to resist such abnormal wind pressures as fifteen or twenty pounds per square foot over the whole floor surface. To meet such a condition with normal unit stresses would require excessively-strong and prohibitively-costly machinery; and it is believed that under such specifications one is warranted in using as low factors of safety as will ensure unit stresses within the elastic limit, for such pressures may never occur with the bridge raised during its entire lifetime.

Such a holding power requires a severity of brake action not only undesirable but dangerous in normal operation; and. therefore, if provision for such holding power must be made, it should be by means of auxiliary brakes, to be used only in case an emergency should arise.

The economic importance of the use, as far as possible, of machinery and apparatus of standard manufacture cannot be too strongly emphasized, not only in view of the lower first cost, but also in the matter of maintenance; because, where special designs are adopted requiring special patterns, those patterns must be stored pending future repairs, which, occurring after long lapses of time, will involve delay in the location of the required patterns, if they can be found at all, and extra expense and time in the reproduction of the parts from them. The designer of the bridge, therefore, should provide ample space for machinery with a view to the kind to be installed, as often the use of special apparatus may be necessitated by the lack of a few feet or even a few inches of room.

The author has noticed a tendency to extreme economy in structural metal at a sacrifice of space required for simplicity of machinery. Structural metal is cheap compared to machinery; and, especially in bascule bridges, the latter involves a large portion of the total cost, hence a very considerable apparent extravagance in structural metal may often result in a reduction of total expenditure, by simplification of the machinery and by provision of space for standard apparatus.

CHAPTER XXXII

POSSIBILITIES AND ECONOMICS OF THE TRANSBORDEUR

MOST of the contents of this chapter are taken from a paper, having the same title, presented to the Institution of Civil Engineers of Great Britain,* but it is supplemented by some later investigations concerning a proposed crossing of the Delaware River between Philadelphia, Pa., and Camden, N. J., on the preliminary economic studies for which the author had previously been retained.

The "*transbordeur*," as it was named in France where it was originated by the noted engineer, Monsieur F. Arnodin, or "transporter bridge," as it is called in England where a number of structures of that type have been built, or the "aerial ferry," as it is termed in the United States, where there is only a single example, is a rather inferior substitute for a low-level bridge. The author prefers to adopt the name "*transbordeur*" not only because of the prior claim of that appellation but also on account of its being shorter than either of the other cognomens; and he is going to take the liberty of anglicizing it hereinafter by omitting to put it in italics. The only excuse for the existence of this type of structure is that its construction is permissible at certain locations where no low-level bridge would be allowed, and where a high-level structure would be unsatisfactory for the crossing traffic. Such conditions exist where the land adjacent to the waterway is low, and where many high-masted vessels have to pass, or where the channel forms the entrance to a harbor of refuge. Under the latter condition, any low-level bridge might prove to be a serious menace to navigation; for it is conceivable that the movable span might get out of order and become immovable for a while during a high wind when vessels are passing through the channel in order to reach the safe harbor beyond.

In comparison with the low-level bridge, the inferior features of the transbordeur are as follows:

First. Its carrying capacity for automobiles during any given length of time is much smaller.

Second. The time necessary for crossing by it, up to the present at least, is much greater.

Third. The costs of both power and labor for operation are higher.

Fourth. While the actual first cost of structure is about the same, or possibly a little less, in respect to general service rendered it is larger.

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* The contents of this paper were used by the Editor of *Le Génie Civil* as the basis of a long editorial, published in two successive issues.

The ordinary transbordeur consists of two towers, an overhead span between them high enough to clear the masts of the tallest vessels, a single track on the span, a car running upon the track, a traveling platform suspended from the car, a fixed platform at each end of the car's travel for unloading and reloading, and approaches to these fixed platforms from the streets. Up to the present time all of the transbordeurs yet built are single-span, single-track, single-carriage, slow-motion structures, consequently their efficiency is low and their use is confined to comparatively narrow waterways.

There are four types of bridge suitable for carrying the cages of the transbordeur, viz., the simple-truss, the continuous-truss, the cantilever, and the suspension. The choice between these will depend almost entirely upon the governing conditions at the crossing.

In "Bridge Engineering" on p. 674 the author in 1916 wrote as follows:

> If the author were ever called upon to design a transporter bridge, he would effect a great improvement by widening the structure so as to provide for a double track, and would carry on it four or more cars. These cars would always travel upon the right-hand track, and would run onto a single track at each end of span where they would discharge and take on passengers. Again, he would use powerful electric motors so as to travel at high speed. By these means, the carrying capacity of the bridge would be multiplied many fold and the time required for transit would be reduced to a minimum; because the intervals between cars could readily be made as small as one minute, requiring only sufficient time to unload and reload the foot passengers and vehicles. The car should be made double deck, the pedestrians being carried above; and the roadway should have a double track, the right one being for the use of a single street-car and the left for two, or possibly three, wagons. At the end of the trip the car would leave first, and the wagons would follow immediately, edging over to the right so as to permit of the ingress of the oncoming car, which in its turn would be followed by wagons to occupy the left-hand side. While the vehicles would be going off and others getting on, the upper deck could easily be emptied of its pedestrians and refilled.

At the time the preceding was written the author did not anticipate encountering at all shortly an opportunity to figure upon a transbordeur of the type described; but in 1918 he was selected by the City of New Orleans as bridge expert to serve as one of the three members of the Board of Advisory Engineers specially appointed to study the governing conditions and to report upon the advisability of bridging or tunneling the Mississippi River at or near that city. In the course of the investigations, which occupied nearly a year, there arose the question of building a combined-highway-and-electric-railway structure connecting New Orleans and Algiers. As the line joining the centers of gravity of these two places is below the center of gravity of the cities' wharves, many ocean-going steamers cross it daily, and there will be a still greater number per diem in the future. For this reason a low-level bridge at this location is inadmissible, but a transbordeur layout would undoubtedly be accepted by the War Department, as the resulting structure would interfere very little, if any, with navigation.

Through the courtesy of the "Bridge or Tunnels Committee" of the "Public Belt Railroad Commission," the author is permitted to utilize for this memoir the results of the transbordeur investigations which he made for the said Board. His design for that transbordeur involved multiple spans, double track, multiple cages, and rapid transit, the speed of travel reaching a maximum of thirty (30) miles per hour. Comparing this with the before-mentioned existing single-span, single-track, single-cage, slow-motion structures, it is evident that the said investigation has revolutionized transbordeur designing by raising the carrying capacity per hour to something like that of the corresponding low-level bridge, when due consideration is given to the time lost by reason of passing vessels.

In the New Orleans investigation it was necessary to make estimates of cost, based on *ante-bellum* unit prices, for both high-level and low-level combined-highway-and-street-railway bridges, notwithstanding the fact that, in all probability, the latter type would be inadmissible; and estimates were added for four transbordeurs, two to carry cages one hundred feet long and the others to support cages of half that length. The results of these computations are given in the following table:

TABLE 32*a*

Type of Structure	Total Cost of Structure Based on *Ante-bellum* Unit-Prices
High-Level Bridge	$5,590,000
Low-Level Bridge	2,660,000
Transbordeur with four long cages	3,160,000
Transbordeur with six long cages	3,250,000
Transbordeur with four short cages	2,450,000
Transbordeur with six short cages	2,500,000

From the preceding table may be drawn the following conclusions:

First. The high-level bridge is more than twice as expensive as the low-level bridge. This agrees with the deductions that can be drawn from comparisons of the estimates for high-level and low-level combined-bridges, and of high-level and low-level steam-railway bridges, which were made for the New Orleans investigation.

Second. The transbordeur with short cages is a little cheaper than the low-level bridge.

Third. The transbordeur with long cages is some 22 per cent more expensive than the low-level bridge, but costs only 58 per cent of the price of the high-level bridge.

The utmost capacity of the transbordeur with six short cages is 40 cages per hour in each direction; and each cage is capable of carrying a fully-loaded street-car, four automobiles, and 250 pedestrians, making 40 cars,

160 automobiles, and 10,000 pedestrians per hour *in each direction.* This estimate is based upon the assumption of there being no interference from river traffic. Of course, the capacity of the transbordeur with six long cages is about twice as great as the preceding figures indicate.

Comparing the economics of the transbordeur with short cages and that with cages twice as long, it is evident that the addition of 30 per cent to the cost of the former will nearly double its transporting capacity; consequently, if the probable demand upon the structure within, say, half a century be greater than its figured capacity for short cages, it might be built strong enough to carry long cages, and be operated with the short ones until such time as the long ones are required. Moreover, it would be practicable to connect two short cages so as to form the equivalent of one long one. Such an arrangement would reduce by about $150,000 the first cost of the most expensive of the four transbordeurs tabulated.

As for the question whether it is preferable to employ four or six cages—it would be economical to adopt four at first and then increase the number to five and finally to six as the traffic augments.

It should be noted that there are several rather-widely-separated crossings of the river which are practicable for the location of the transbordeur, consequently it might be economic to build the cheaper structure, and later, when its capacity is nearly reached, construct another some distance away. Two widely-separated structures of a certain capacity would be far more serviceable to the public than a single structure of double that capacity.

The transbordeur may prove in years to come the ideal type of structure to provide for pedestrian, vehicular, and street-railway traffic across the river near the heart of the City; because the high-level bridge is so expensive and involves such a great climb and such long detours that it is really out of the question; and a low-level structure below the up-stream city-limits would offer too much obstruction to navigation. Owing to its long spans and its great vertical clearance, which extends from levee to levee, the transbordeur would cause less obstruction to navigation than any other possible type of structure.

While the cost of operating a transbordeur may figure out to be greater than that of operating the movable span of a low-level bridge or the passenger elevators of a high-level bridge, if one will include the cost of the animal power, auto power, and trolley power expended in traversing a low-level bridge (excluding, of course, the approaches), the economics may prove to be reversed.

The character of construction and the *modus operandi* of operation of the suggested transbordeur are illustrated in Fig. 32*a*, from which it is seen that the distance between centers of levees at the selected crossing is 2,250 feet, divided as follows: At the center of the river there is a tower

200 feet long transverse to the stream; and, on each side of this, a 700-foot span with a cantilevered end extending out 400 feet beyond a supporting pier, on which is a rocker-bent that is stiffened laterally by means of two triangular-braced frames. The inclined legs thereof connect to the ends of the cantilever arms of a very deep steel-girder, encased in concrete, that joins the two concrete cylinders of which the pier is composed. There is a similar arrangement of bracing at each of the piers near mid-channel. The outer end of each of the 400-foot spans rests on a roller bearing directly above the end columns of a steel-trestle approach, being anchored thereto so as to prevent any uplift. Of course there can be no bracing in the central portion of any one of the braced towers, because the ferry cars or cages have to pass through that space. The trusses are 100 feet deep; and they are braced across between chords in both horizontal and vertical planes, so as to stiffen them thoroughly. The long-type cages, of which there are six, are about 100 feet long and 25 feet wide from out to out. They will carry on one side of their lower deck a line of track long enough for two street-cars, and alongside of these there will be space for five or six wagons, or eight automobiles, or five motor trucks. The cage is to be double-decked, the upper deck being for pedestrians and covered with a roof; the sides are to be open to permit the wind to blow through, but arrangement must be made to cover the upper portions of them, occupied by the pedestrians, in case of rainy or cold weather. The cages are suspended by rigid frames braced on all four faces—completely at the sides, and at the ends to within about 8 feet of the top of the upper deck. These cages are hung by trucks, the wheels of which roll on rails supported by the bottom flanges of stringers. In order to avoid delay, the cars are always to be suspended from overhead, and are not to be carried by tracks on the approaches at the level of the platform. Near the outer end of each trestle, there is a pair of independent travelers running on overhead transverse tracks.

At each end of the structure there are four pockets, two for operation and two for storage. The latter have overhead stringers for carrying the wheels of the traveling cages.

The *modus operandi* is as follows:

At the end of the structure shown on the left in Fig. 32*a*, a traveling cage comes along "Track A" and passes into "Traveler A," which is immediately moved outward into "Pocket A." At once "Traveler B" is moved so as to face "Track A," from which it receives the next incoming cage, and then moves into "Pocket B." Meanwhile the street car, the vehicles, and the pedestrians of the first cage have passed out to the left, as indicated by the arrow, and the cage is again filled by a car, vehicles, and pedestrians coming in from the right, as shown by the arrow adjacent to the curved center-line of street-railway track. Then "Traveler A" moves over to face "Track B," and the cage with its contents starts back across the structure, immediately after which "Traveler A" moves over

to "Track A" to receive the next incoming cage. Then "Traveler B," its cage having been emptied and refilled, as indicated by the arrows, passes over to "Track B," and its content starts back across the structure. Finally "Traveler B" moves over to "Track A" to receive another incoming cage, "Traveler A" having meanwhile received an incoming cage and moved over to "Pocket A." This completes a cycle of traveler operations at the left end.

At the other end, the first returning cage enters "Traveler A′," which immediately moves into "Pocket A′," where the car, vehicles, and pedestrians pass out to the left, as indicated by the arrow adjacent to the center-line of street-railway track; then the re-loading is done from the right, as indicated by the other arrow. Meanwhile, "Traveler B′" has moved to "Track B," received the second cage, and carried it to "Pocket B′," where it is unloaded to the left and reloaded from the right, as shown by the arrows. After the cage in "Traveler A′" is reloaded, the said traveler moves over to "Track A" and lets the cage pass out for another trip. Then "Traveler A′" moves over to "Track B" to receive the third cage, which it takes to "Pocket A′," and "Traveler B′" with its reloaded cage passes to "Track A" to discharge its contents. This completes a cycle of traveler operations at the right end.

As the rush traffic of the morning and evening hours is reduced, one or more cages can be run to the storage pockets, where they can be lubricated, cleaned, and put into good order; and at night a single cage can be operated by filling all four of the storage pockets and one of the other pockets with cages.

It will be noticed that while the automobiles and the other vehicles always head in the same direction, they do not have to back out of the cages. It would be practicable to let the said vehicles head in either direction by putting a double track on each cage and turning the cars and other vehicles out quickly to the right on the approaches. This would necessitate enclosing in a "corral" all incoming vehicles until the cage is emptied. There does not appear to be much choice between these two methods of operation.

Fig. 32*a* indicates the method of caring for pedestrians. They pass up the stairways shown at the ends to covered platforms alongside the storage pockets, where they await the arrival of the cage; and they would not be allowed to enter the latter until after its incoming load of pedestrians had passed off by the adjacent exit-stairway. There could, therefore, be no clashing of incoming and outgoing pedestrians; and the approaches to the entering stairway could be arranged so as to avoid any interference between the pedestrians and vehicles of all kinds.

Assuming a maximum attainable velocity of thirty miles per hour, which may not always have to be utilized, and with only four cages and two travelers operating, the following time schedule was figured, based upon an allowance of three minutes for a cage to cross the river:

Time in Minutes	Trav. A	Trav. A′
0	Cage 1 enters Trav. A	
¼	Cage 1 enters Pocket A	
1¼	Cage 1 leaves Pocket A	
2	Cage 1 enters Track B	
2½	Cage 2 enters Trav. A	
2¾	Cage 2 enters Pocket A	
4¼	Cage 2 leaves Pocket A	
4½	Cage 2 enters Track B	
5	Cage 3 enters Trav. A	Cage 1 enters Trav. A′
7	Cage 3 enters Track B	Cage 1 enters Track A
7½	Cage 4 enters Trav. A	Cage 2 enters Trav. A′
9½	Cage 4 enters Track B	Cage 2 enters Track A
10	Cage 1 enters Trav A	Cage 3 enters Trav A′
12	Cage 1 enters Track B	Cage 3 enters Track A
12	Cage 2 enters Trav A	Cage 4 enters Trav. A′
14½	Cage 2 enters Track B	Cage 4 enters Track A
15	Cage 3 enters Trav. A	Cage 1 enters Trav. A′

In the above schedule it will be noted that during the first four and a half minutes both the major and the minor operations are noted, but that in the next ten and a half minutes, for the sake of brevity, the latter are omitted.

The above layout gives two-and-a-half minute service in each direction.

With six cages and four travelers operating, the following time schedule was figured, based upon an allowance of two and a quarter minutes for a cage to cross the river:

Time in Minutes	Trav A	Trav. A′
0	Trav. A faces Track A	
0	Cage 1 enters Trav A	
¼	Trav. A enters Pocket A	
1¼		Trav. B faces Track A
1½		Cage 2 enters Trav. B
1¾	Trav. A leaves Pocket A	Trav. B enters Pocket B
2	Trav. A faces Track B	
2¼	Cage 1 leaves Trav. A	

Time in Minutes	Trav. A	Trav. B	Trav. A′	Trav. B′
2¾	Trav. A faces Trk A			
3	Cage 3 enters Trav. A			
3¼	Trav. A enters Pock. A	Trav B leaves Pock. B		
3½		Trav B faces Trk B		
3¾		Cage 2 leaves Trav. B		
4¼		Trav. B faces Trk A	Trav A′ faces Trk. B	
4½		Cage 4 enters Trav. B	Cage 1 enters Trav A′	
4¾	Trav. A leaves Pock. A	Trav. B enters Pock. B	Trav. A′ enters Pock. A′	
5	Trav. A faces Trk B			
5¼	Cage 3 leaves Trav A			
5¾	Trav. A faces Trk. A			Trav. B′ faces Trk. B
6	Cage 5 enters Trav. A			Cage 2 enters Trav. B′
6¼	Trav. A enters Pock. A	Trav. B leaves Pock B	Trav. A′ leaves Pock. A′	Trav. B′ enters Pock. B′
6½		Trav. B faces Trk. B	Trav. A′ faces Trk. A	
6¾		Cage 4 leaves Trav. B	Cage 1 leaves Trav. A′	
7¼		Trav. B faces Trk A	Trav A′ faces Trk. B	
7½		Cage 6 enters Trav. B	Cage 3 enters Trav A′	
7¾	Trav. A leaves Pock. A	Trav. B enters Pock. A	Trav. A′ enters Pock, A′	Trav. B′ leaves Pock B′
8	Trav. A faces Trk. B			Trav. B′ faces Trk A
8¼	Cage 5 leaves Trav. A			Cage 2 leaves Trav. B′
8¾	Trav. A faces Track A			
9	Cage 1 enters Trav. A			

This layout gives one-and-a-half minute service in each direction.

The preceding method of handling the cages by means of travelers was not designed *instanter*, but is the result of a gradual development through a series of tentative studies. The first idea that occurred to the author was to carry the cages by the suspension system around a loop of large diameter at each end of the route, but this necessitated a very awkward detail, troublesome to operate, consisting of a pair of movable platforms to permit cars and other vehicles to enter or leave the cage at the ends. This alone was sufficient reason for condemning the method; but there was a still-more-important one, viz., that a breakdown of one of the traveling cages would tie up the whole line. This contingency might have been provided for by having outside of each loop a traveler leading to a "hospital" pocket; but that detail also would have been awkward and troublesome to operate, hence the loop-system project was soon abandoned.

Next came the idea of using switches at the approaches, but it was evident at once that overhead ones were impracticable on account of the necessity for cutting through the supporting girders, hence they would have to be located beneath the cage, which would then require trucks below as well as above, and the support would have to be changed at very short intervals between the upper and the lower systems. This method, too, was awkward, requiring both time and an additional expenditure of power to make the change, consequently it, also, was soon abandoned.

Next came the scheme of operating the two tracks as entirely independent units, but adjusting the times for starting so as to have comparatively regular intervals between cages in each direction. This scheme involved the idea of the lateral travelers running on transverse tracks below, with the cage still supported from above by longitudinal tracks which would be a continuation of those on the span, the interval between being so small as readily to be jumped by the wheels.

The layout for this method of operation is shown in Fig. 32*b*, from which it is seen that near the outer end of each trestle-approach there is to be a pair of independent, double-chambered travelers running on transverse tracks, the extreme motion of each traveler being about 25 feet. These travelers are entirely disconnected from the top of the trestle, but lie up very close to it. Beyond each traveler is a stationary pocket (large enough to receive a cage) braced at the sides, but open at the river end for the full height, and open at the shore end near the bottom high enough for the ingress and egress of vehicles and pedestrians.

The *modus operandi* is as follows: As shown at the right-hand side of Fig. 32*b*, looking from the river, the traveler is at its outer position, all three of the cage-receptacles being vacant. Cage 1 arrives and runs into the inner chamber of the traveler, which is then moved inward so as to allow the contents of the cage to be unloaded and a new load to be taken on. While this is occurring, Cage 2 arrives and passes across the outer chamber of the traveler to the pocket beyond, and from there the unloading

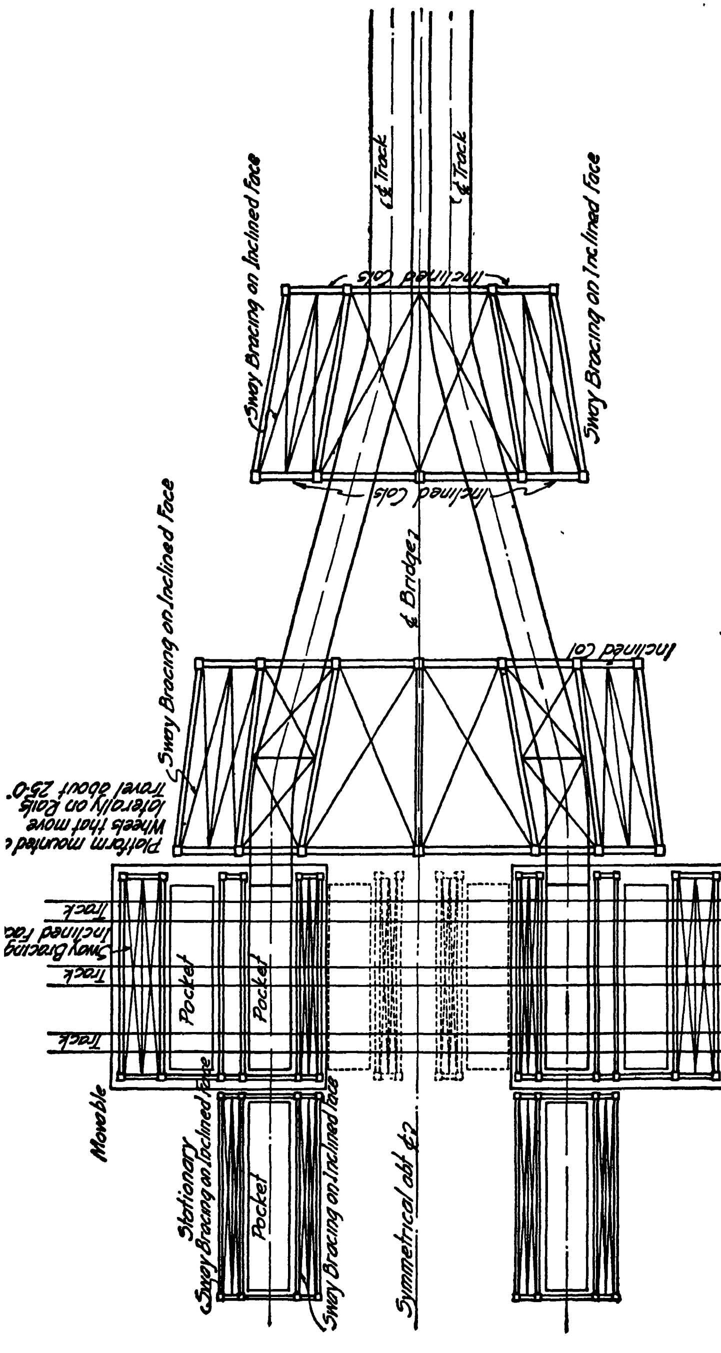

FIG. 32b. Second Preliminary Study for Proposed New Orleans Transbordeur.

and reloading take place. Meanwhile Cage 3 arrives and runs into the outer chamber of the traveler, after which the latter is moved to its outer position, so as to allow Cage 3 to unload and reload. This brings Cage 1 into position to begin its return journey; and, after the proper interval, Cage 2 also starts back. Next, when Cage 3 is loaded, the traveler moves inward so as to permit that cage to start. This operation is repeated at the other end of the structure.

The operation of the other track is similar; and, as previously indicated, the times of starting the two groups of cages are adjusted so as to make the intervals between all cages as nearly uniform as practicable.

The time of travel was figured as follows for the round trip of one cage: Assuming that the said cage is in its pocket, loaded and ready to start, then

Trip across structure 3,100 ft. @ 30 miles per hour = 1.17 min. or, to allow for acceleration and retardation, say..........	1.50 min.
Wait of two one-minute intervals between cages, which time will suffice for one lateral transfer and the unloading and the reloading of the cage............................	2.00 min.
One side shift of traveler..............................	0.25 min.
Return trip	1.50 min.
Two intervals of one minute each between cages............	2.00 min.
One side shift of traveler..............................	0.25 min.
Total time for one round trip.................	7.50 min.

The number of round trips per hour for each cage will be $60 \div 7.5 = 8$; and, as there are six cages, there will be altogether 48 round trips made on the two tracks.

The intervals between cages on each track in the same direction will be one min., one min., and five and a half min. By starting the first cage of the group of three on Track "B" one and three-quarters minutes after the third cage on Track "A" has left, the intervals between starts at either end would be as follows:

1 min., 1 min., $1\frac{3}{4}$ min., 1 min., 1 min., $1\frac{3}{4}$ min., 1 min.,
1 min., $1\frac{3}{4}$ min., 1 min., etc., etc.,

Under the assumption of cars one hundred feet long, the capacity of the structure per hour in each direction was estimated to be 48,000 persons plus a large but unknown amount of freight. Assuming also that one-fourth of the people of Algiers would go to New Orleans every day, and that one-half of this number would pass over in a space of three consecutive hours, this structure would comfortably accommodate two cities, each having a population of a million. On the basis, however, of the accommodation of passenger automobiles and motor trucks in the proportion of those operating in the City of New Orleans, viz., three automobiles to each motor-truck with one auto-vehicle to each 33 inhabitants, and assuming

that for twelve hours per day the cages would run full and the other twelve hours only half full, the proposed transbordeur would serve a population of about 475,000 in Algiers, or, say, 400,000 to allow for interruptions from river traffic. Were the short cages used instead of the long ones, the population that could be served would be 200,000, showing that, for many years to come, even with the short cages, the structure would have ample transporting capacity, because Algiers at present is only a small community.

The next improved layout is shown in Fig. 32*c*, which indicates that the number of transverse travelers had been increased to six and that the travel on each track is in one direction only. In this case the travelers run on tracks beneath them; but after the drawing had been made, some figuring showed that it would be more economical to suspend the said travelers from tracks overhead, as indicated in the layout of Fig. 32*d*. In both of these layouts, by using only four cages, one traveler at each end, and $1\frac{3}{4}$ minutes for crossing the river, a service of $2\frac{3}{4}$-minute intervals could be obtained, provided there were no interference from river traffic. With three travelers at each end and five cages, the interval would be reduced to $2\frac{1}{4}$ minutes; and by adding another cage it could be made as small as $1\frac{3}{4}$ minutes. As indicated previously, the final layout with only two travelers at each end gave $2\frac{1}{2}$-minute intervals with four cages and $1\frac{1}{2}$ minute intervals with six cages, thus reducing both the total first cost and the expense of operation.

It is difficult to compare the operating capacity of a transbordeur with that of a low-level bridge of the same general cross-section, but it might be well to make the attempt, assuming as an equivalent low-level structure the layout adopted for New Orleans, viz., the author's standard city-bridge, having a clear roadway of 42 feet with two street-railway tracks at the middle and two exterior sidewalks, each of eight feet clear width.

Assuming three lines of pedestrians per sidewalk with intervals of three feet in each line, which represents as dense a crowd as could cross comfortably, and a speed of three miles per hour, makes 15,840 foot-passengers per hour in each direction, while the transbordeur with 50 ft.-cages could carry about two-thirds of that number. In respect to a combination of street cars and automobiles, the average speed when the roadway is at all crowded will not exceed twelve miles per hour, and in such a case the distance between autos would be about sixty feet. It would be seldom that there would average more than one street car per minute on each track, and in that case there would not be more than one automobile in 120 ft. traveling along the street-car space; hence there would be $1080+540=1620$ automobiles passing in each direction per hour or ten times as many as the 50 ft.-cages would carry. There would be about one and a half times as many street-cars traversing the bridge as could be carried by the transbordeur with the short cages. Were there a horse-drawn vehicle or two on one side of the roadway, especially a heavily loaded one, the average speed for the automobiles would quickly reduce to six miles per hour or less, because the existence of occasional street-cars, as well as the automobiles

occupying the street-car space, would prevent the other automobiles from passing quickly the slow vehicles. This cutting of the speed in two does not by any means halve the capacity of the bridge, but it certainly reduces it materially, possibly to 75 per cent; because the space between vehicles ould reduce with the decrease in velocity. The greater the number of

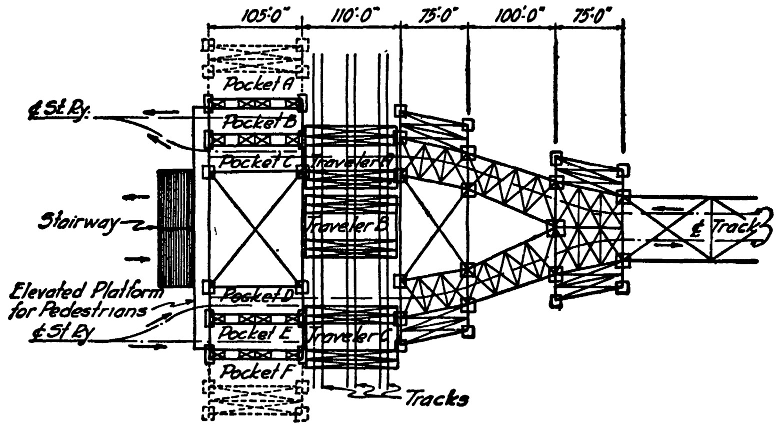

FIG. 32c. Third Preliminary Study for Proposed New Orleans Transbordeur.

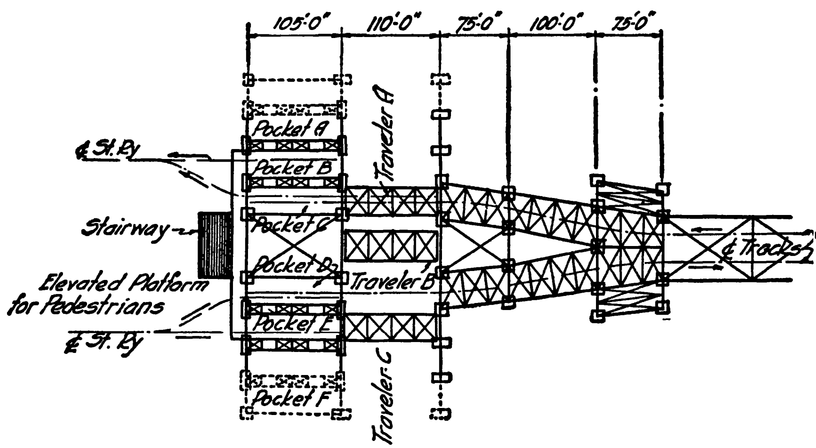

FIG. 32d. Fourth Preliminary Study for Proposed New Orleans Transbordeur.

horse-drawn vehicles on the bridge the slower would be the average speed of the procession in the other line; and with a great many of them on the roadway, the automobiles would naturally confine their travel mainly to the street car line and adjust their speed to that of the cars.

It is evident, therefore, that, before designing any transbordeur, one should study carefully both the present and the probable-future proportions of all kinds of travel and adjust the design upon the principle of "the

greatest good for the greatest number." The limitation of efficiency will generally be found to be the capacity for transferring automobiles; and this can be greatly augmented by making the cages three-decked, taking care of the street-cars and the auto-trucks on the lower one, the automobiles on the middle one, and the pedestrians above.

From the preceding reasoning it may be concluded that in the comparison between a double-track transbordeur with short-cages and a standard, 60 ft.-wide, highway bridge, the former is two-thirds as effective for both pedestrians and street-cars, and only one-tenth as effective for automobiles. If the 100 ft.-long cages were used and only one street-car were carried per cage, the rest of the space being occupied by automobiles, the preceding ratios would be about one and a third, two-thirds, and one-third; or, if the long cages carried two street-cars each, these figures would be one and a third, one and a third, and one-fifth. The comparative efficiencies of the two structures would, consequently, be dependent upon how the traffic is divided between pedestrians, street cars, and automobiles. For the first two the transbordeur can readily be made but little lower in efficiency than the low-level bridge, but for auto-traffic it will always be found decidedly inferior; and, as that traffic is on the increase in both amount and importance, it must inevitably be concluded that, in respect to general carrying capacity, the transbordeur is never as satisfactory as the said low-level bridge.

The making of this comparison was advisable, although by no means essential, because there should never be any choice between a bridge and a transbordeur for any proposed crossing. As stated at the beginning of this chapter, if the low-level bridge is permissible, it should be adopted; but, if not, the transbordeur should be used as a *pis aller*. The author's object in making the attempted comparison was to confirm in a general way his *a priori* conclusion.

As previously indicated, there will not be many occasions for the building of the transbordeur; and the conditions of traffic, navigation, river-width, and property for approaches are so variable that each case will require a thorough and systematic compilation of them all, for both the immediate and the distant future, and an exhaustive study of the question how best to compromise between conflicting interests and to develop in general the greatest possible efficiency.

The question might sometime arise as to whether a transbordeur or a high-level bridge would be preferable for the crossing of a waterway navigated only by river steamers, for which the Government's clearance-requirement generally varies from 50 to 60 feet above high-water elevation; consequently, it would be well to know in advance the approximate ratios of cost of a transbordeur to the various costs of the corresponding high-level bridges having differing vertical clearances. For this purpose Fig. 32*e* was prepared by using as a basis a slight modification of some of the results of the computations for the New Orleans Bridge study. That figure gives

for the crossing at that place, when the pier-locations and span-lengths are constant, the ratios of costs of fixed-span bridges and their approaches for all clearances above high water up to 200 feet, using as the comparing unit the cost of the bridge having a clearance of only ten feet, which is as small as generally is permissible. Curiously enough, the costs of the three bridges specially figured for the diagram make the record a straight line. The cost of the transbordeur with short cages in this case is almost equal to that for a fixed-span bridge having a clearance of ten feet, as given by the diagram; and that for a clearance of 60 feet is shown to be 1.28 times that for the lowest structure and, therefore, also 1.28 times that for the transbordeur. In view of the comparatively small difference in cost between

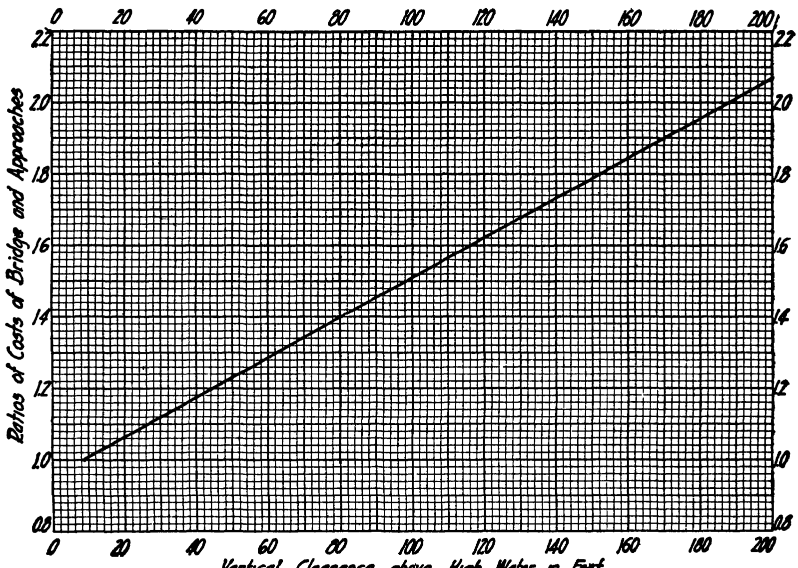

FIG. 32e. Ratios of Costs of Fixed-Span Bridges and their Approaches for Various Clearances Above High Water.

transbordeur and fixed-span bridge, and of the facts that a climb of 65 feet above the water is by no means prohibitory for any class of vehicle, even the horse-drawn,—that the carrying capacity of the bridge is generally greater than that of the transbordeur for both pedestrians and street-cars and always much greater for automobiles,—and that horse-drawn vehicles are fast becoming obsolete,—the deduction may be made that, for crossings with moderately-high vertical-clearance, the fixed-span bridge is decidedly preferable to the transbordeur. It is true that this general inference has been drawn from a single case, and on that account may not be satisfactory to some engineers; but the author is of the opinion that, were similar reasoning applied to any other practical case, a like conclusion would be reached.

Before closing this chapter the author has decided that it will be well to take two pending cases from his own practice and indicate how they should be solved. One is that of the crossing of Havana Harbor, Cuba, so as to develop a tract of beautifully-situated and almost-entirely unoccupied land about five miles long and one mile wide paralleling the coast line. Some seven years ago the author spent considerable time and money on the development of the project to build a high-level, cantilever, highway-and-street-railway bridge over the entrance channel near the inner end of Cabaña Castle at a location quite close to the place where the said channel expands into the body of the gourd-shaped bay forming the anchorage-ground of the harbor. Just as the project was about to materialize, the war in Europe began; and ever since then it has been impracticable to raise money for any large enterprise not directly connected with war work. However, the time is probably close at hand when it will be advisable to try to revive the scheme, because sooner or later a bridge of some kind will certainly be built across the channel so as to permit the city to expand in the one possible direction that will permit of the building of fine residences within quick reach of the business center. The old layout, shown in Fig. 32*f*, necessitated the carrying of all traffic from an elevation of five feet above water level on the city side by means of a spiral approach and passenger elevators to an elevation of two hundred and four feet above the same, and then down to an elevation of about one hundred and fifty feet at the entrance to the North approach.

About a year ago, it struck the author that it might be better to build a double-track transbordeur quite close to the mouth of the harbor and to carry the traffic across at an elevation of about eighty feet above the water. Near La Punta, the outermost land on the City side of the channel, there is a large area of open ground which rises gradually to an elevation of twenty-five or thirty feet some four or five blocks back from the water; and at the other side of the harbor, between Morro and Cabaña Castles, the elevation of the land is much lower than it is farther in. By starting with a trestle approach near the South end of the said open ground and rising on a four per cent grade, more or less, to within about five hundred feet of mid-channel, it would be practicable to attain the elevation just mentioned for the cage deck; and on the other side a short trestle with an easy up-grade would lead to the general surface of the ground back of the two castles. The length of span over the channel would not be more than 800 feet; and, possibly, it might be a hundred feet shorter, thus making the travel of the cages only 800 or 900 feet. On that account it would not be economical to adopt the transverse travelers, because the time required for crossing is so short. It would pay better to widen the structure and put in another track or two, should the prospective traffic warrant it, especially as, on account of the great height of the overhead span, a large width of structure is necessary for resisting properly the overturning effect of the wind pressure, which during tropical storms is likely to be excessive. In order to

FIG. 32f. Proposed Bridge Across the Entrance Channel to the Harbor of Havana, Cuba.

secure a large capacity, it would be advisable to make the cages three-decked, and possibly longer than fifty feet. Again, in order to save time in unloading and reloading, the street-cars leaving the city side could take the lower deck and the automobiles and other vehicles leaving the same could occupy the middle deck, the pedestrians, of course, always using the upper deck. Starting from the other side, the automobiles could occupy the lower deck, and the street-cars the middle deck. By this arrangement the climbing of the comparatively steep grade at the North end from the lower-deck level would be done by street-cars only, and time would be saved by holding the automobiles and horse-drawn vehicles in a corral until after the street-car has left clear both the cage and the portion of the approach close thereto. Similarly, the street-car would be held in a corral until after all the other vehicles have gotten out of the way. Barring interference from navigation, each cage could make a round trip in about 3½ minutes, which would give 17 round trips per hour, or 34 trips per hour in each direction by the two cages. A 60 ft.-cage would carry two street-cars, eight auto-vehicles, and 300 pedestrians, making 68 cars, 272 auto-vehicles, and 10,200 pedestrians per hour in each direction. Allowing for interruptions from navigation, which would not be serious, because of the great clearance beneath the cages that would permit a large proportion of passing craft to go by without stopping operation, it would be legitimate to count upon a maximum limit of 60 street-cars, 240 automobiles, and 9,000 pedestrians per hour in each direction. Deducting liberally for service to outlying towns, the remaining capacity would be great enough to provide for a population of at least 75,000 persons, or 15,000 per square mile, which is fairly dense for a residence district.

Fig. 32*g* shows in skeleton outline the elevation of this transbordeur span with a short portion of each of the adjacent approaches, also a cross-section of the structure at mid-span. From this drawing it will be seen that, in reality, there are two distinct, parallel, cantilever bridges located a short distance apart and connected to each other by cross-girders and bracing, leaving ample space between for the safe passage of the two cages; that the latter pass shoreward beyond each main pier a distance equal to one panel-length of the anchor arm, connecting to an approach deck that is suspended from the other two panel points of the said anchor arm; that all the panels of the entire structure are of equal length, there being three of them in each anchor arm, five in each cantilever arm, and six in the suspended span; and that the outlines of all chords are parabolic curves. Each main pier consists of four pneumatic cylinders sunk to bed rock, each cylinder being surmounted by a truncated concrete cone, and the three spaces between the four cones being filled with thin, reinforced-concrete walls. Each anchor-pier shaft consists of a mass of concrete supported upon a concrete base that rests on piles. The author believes that the bold, curved outlines of the trusses, in spite of the horizontal line of girders between the twin spans, would make an æsthetic structure that would be

in keeping with the commanding position which it would occupy at the entrance to one of the most beautiful harbors in the world.

The other case, as indicated at the beginning of this chapter, is that of a proposed suspension bridge, having a single span of 1,750 feet which clears the entire width of river between the established harbor lines, to join the cities of Philadelphia and Camden.

Quite lately the author was struck with the idea that a transbordeur might serve all the traffic at the crossing and save considerable money as compared with a bridge, consequently he had his office prepare a layout, using a three-deck cage to carry street-cars, automobiles, and pedestrians; but a short preliminary economic study showed that the expense involved would be too great, consequently he prepared a two-deck layout that does not carry street-cars. These would run around a loop on each approach and would discharge and take on passengers close to the loading places of the cages. An estimate of cost was made for this layout based on the then-ruling prices of materials and labor, the grand total, excluding right-of-way, property damages, and interest during construction, amounting to $12,000,000. Then a layout was made for a bridge and its approaches, and a similar estimate of cost was prepared, amounting to $13,400,000.

As these two amounts differ so little, it was at once concluded that for this crossing there would be no real economy in adopting a transbordeur. The ratio of the said amounts is about 0.9, while the corresponding ratio for the New Orleans study was only 0.58.

From these figures it may be concluded that a long-span-suspension layout does not accommodate itself to the carrying of transbordeur cages as economically as does a layout of several shorter continuous spans. The explanation of this fact is that the piers for the two cases are about alike, that the approaches of the transbordeur are nearly but not quite as expensive as those of the bridge, and that while there is a large saving in the combined costs of superstructure and anchorages, it is largely offset by the cost of the cages, travelers, and pockets. In the New Orleans investigation the substructure of the transbordeur would cost but little more than one-half of that for the corresponding bridge, and the cost of the approaches to the former is a bagatelle when compared with that of those to the latter; but the total steelwork for the river spans, towers, bents, cages, travelers, and pockets of the transbordeur exceeds in value that of the river spans of the bridge.

Conclusion and Recapitulation

The title of this chapter indicates that it is intended to show both the possibilities and the economics of what in slang parlance might be termed the "glorified" transbordeur, i.e., the existing type of transporter bridge expanded and enlarged many fold so as to accommodate it to wide rivers, high carrying capacity, and rapid transit. Some of its possibilities have

been shown, but probably not all of them; and as for the economics, it has been proved to cost less, even in extreme cases, than a very-high-level structure, and about the same as a low-level structure; but that, strictly speaking, it cannot be said to be in economic competition with either type; because when the conditions really call for the consideration of a transbordeur, a very-high-level bridge would generally necessitate too great a climb for the traffic, and a low-level bridge would be barred because of its interfering too much with the paramount interest of navigation. The Philadelphia-Camden case is apparently an exception to this rule, but it must be remembered that its vertical clearance is only 135 feet, while that for the proposed crossing at New Orleans is 175 feet, and that the extra forty feet of height make a great difference in the costs of the approaches.

CHAPTER XXXIII

ECONOMICS IN CONTRACT-LETTING

AT first thought one might be inclined to claim that contract-letting is not a matter of economics, but a little reflection will soon convince him that it certainly is, because if contracts be badly drawn, or if the *modus operandi* of compensating the contractor and his workmen be faulty, the work of construction will assuredly cost more than it would under ideally perfect conditions. During the last few months the author has been writing a series of papers on contract-letting and profit-sharing, culminating in a lengthy discussion of a paper by Mr. Ernest Wilder Clarke, published in the August, 1919, *Proceedings* of the American Society of Civil Engineers, the said discussion appearing in the October-November-December *Proceedings*.

As this discussion is in reality a complete treatment of the subject, and was intended as such (although for good and sufficient reasons presented as an adjunct to another paper instead of being offered as a separate memoir), it is here reproduced practically *verbatim*.

The importance of the subject of this paper, to owners, contractors, and the entire American nation, cannot well be exaggerated; for, until there is reached a satisfactory compromise between owners on the one hand and contractors on the other concerning the vital questions of contract-letting and profit-sharing, the business of the country will fail to recuperate to the extent that it should at this critical period in the readjustment of all constructional activities—which activities were so fundamentally upset throughout the whole world more than five years ago by the advent of the World War. The opportunity now within easy reach of the American people to secure the bulk of the world's trade is unique; but, apparently, our leaders in diplomacy, manufacture, shipping, business, and finance are either unaware of its existence or indifferent about taking advantage of their good fortune. The Latin-American nations are now knocking at our door asking to do business with us and begging us to lend them money for the development of their as-yet-almost-virgin lands, mines, and water powers, and their other more or less embryonic resources; and the leading peoples of Asia, Africa, Australasia, and even Europe are to-day much more willing to enter into business relations with this country than they have ever been before. They all recognize that, for the present at least, the European countries are in no condition to lend money, being

themselves borrowers and having so many demands at home for their time, attention, and capital that they are unable to give consideration to the needs of any nation but their own; and the entire civilized world is well aware of the fact that, in spite of our vast war debt, we are today the most wealthy of all peoples.

These conditions—unfortunately, as far as we are concerned—will not last indefinitely; and if we are so short-sighted as to neglect to seize the golden opportunity which is not only easily within our reach, but which is actually being tendered to us and almost forced on our acceptance, it will not be long before both England and Germany will re-secure the grip on the world's trade which they possessed in *ante-bellum* days.

Prevention of Progress. One of the greatest stumbling blocks in the pathway of our nation's advancement is the conflict between labor and capital; and until it is removed the wheels of progress will be clogged, and the present paralyzation of all great peace industries will continue to exist—possibly in even worse form than it does today. Again, the main issues between labor and capital are these questions of contract-letting and profit-sharing. If they were once settled to the satisfaction of all concerned—bankers, manufacturers, contractors, and workmen—all other minor differences would quickly be adjusted. Such a settlement is perfectly feasible; and the possibility of its speedy accomplishment is neither a Utopian conception nor an idle dream.

Primarily, the men who do the work must have a substantial share in all net profits of manufacture and construction; but the way for them to obtain it is not by the organization of strikes nor by seizing the plant and running the business of the manufacturer or the contractor. The uneducated workman is no more fit to manage business and to handle industry than the sedentary office man is to undertake the physical labor of the workman—in fact, much less so; because it would generally be totally impracticable to educate the illiterate workman up to a state of efficiency which would enable him to undertake business management and finance, while, in most cases, in a comparatively short time, the office man's muscles could be developed sufficiently to enable him to endure the physical stress of the workman's job. Nothing of value can be accomplished by mob rule, as the pending subsidence of the present wave of Bolshevism will soon prove.

Labor and Capital. Although it is certainly true that the laborer cannot succeed independently of the business man, it is equally true that the latter cannot accomplish much without the aid of the capitalist, hence it behooves business men and financiers to come quickly to a friendly understanding and agreement. In times past, the capitalist secured and exercised a strangle hold on the promoter, the manufacturer, and the contractor, often forcing them to turn over the lion's share of their profits as compensation for the use of money in the development of their undertakings; and these men in their turn endeavored to even up matters by

getting their pound of flesh out of the workman by compelling him to labor long hours for meager compensation. Today, the laboring man is beginning to come into his own; but if he unwisely takes too great advantage of his growing power, he will "kill the goose that lays the golden eggs," and this will ruin his chance of securing comfort and happiness for himself and his family.

The first step requisite for quieting the existing widely spread popular unrest and returning to normally prosperous conditions is to bring together, so that they may operate in harmony, the financier, the employer, and the laborer; and this must be accomplished primarily by establishing some method of contract-letting and profit-sharing which will be just and equitable to all parties interested. For some time, the author has been endeavoring to formulate and develop, through communications to the technical press, an ideal method of accomplishing this desideratum; and later herein he will indicate clearly of what it consists. First, however, he will discuss in detail not only the suggestions offered by Mr. Clarke in his timely paper, but also those of various engineers and political economists who have of late been treating the matter in print.

Various Methods of Contract-Letting. The common ways of contract-letting are the following:

A. Lump sum for complete construction.

B. Schedule rates for all materials in place.

C. Actual cost plus a percentage, with some kind of allowance for overhead expenses.

D. Actual cost of labor and materials plus a percentage, with no allowance for overhead expenses.

E. Actual cost plus a lump sum, with some kind of allowance for overhead expenses.

F. Actual cost of labor and materials plus a lump sum, with no allowance for overhead expenses.

G. Actual cost plus a profit based on small unit prices agreed upon in the contract, as advocated of late by Mr. G. H. Hailey.

H. Various methods of profit sharing between the client and the contractor.

Method A generally appeals best of all to the client, because it fixes in advance what the construction is going to cost him, unless, perchance, there are important variations in the estimated total quantities of materials, due to lack of sufficient preliminary information or to the encountering of conditions that could not well have been foreseen. It is by no means as satisfactory to the contractor, however, who has to run the risk of being actually out of pocket on the job, in addition to the loss of personal time and effort.

Method B is quite an improvement on Method A, in that the contractor does not have to guarantee the correctness of the estimated quantities of materials; but the prime objection to Method A holds good for Method B,

as the latter does not prevent the contractor from losing money heavily on the venture.

In the days of hard times, when contractors are willing to take work at low figures, and even below cost, in order to keep their force together, the public, in general, especially as represented by companies and municipalities, is prone to take advantage of them by insisting that work be let by the lump sum, and by throwing on the unfortunate "successful bidder" not only the risk of loss from rising prices of materials and labor and from unforeseen contingencies, but, also, in many cases, from excess of quantities above those given in the specifications. This is accomplished by inserting in the latter a most unjust clause compelling each bidder to verify for himself both the quantities stated and the character of the conditions described. The bidders, hungry for work, accept this clause without comment, but with the mental reservation that, in case of hard luck, they will, by some means or other, obtain extra compensation, even if they have to carry the controversy into the Courts.

In nineteen cases out of twenty, it is unjust to bidders to ask them to name a lump-sum compensation for doing work, unless provision is made for a variation in the quantities of materials on which they tender. If provision is arranged for such variation, the method of letting is no longer that of the "lump sum," but reduces to a modification of that of "unit prices."

As before indicated, the latter method is certainly the more logical, and yet it is far from being entirely fair to the contractor; because, although it provides against loss through excess above the estimated quantities of materials, it leaves him open to the possibility of still greater loss through changing prices, onerous unanticipated conditions, and disastrous happenings beyond his control. Up to a certain point, the client is the proper party to assume the principal risks inherent to the work, provided that the adverse happenings are really unavoidable by the contractor, and that the latter takes every reasonable precaution against disaster or loss. And yet the contractor should not be altogether relieved from the possibility of loss due to hard luck, because such misfortune is often caused by his dilatoriness, which carries the work into an unfavorable season for field operations. The idea, promulgated of late by certain writers, that it is unjust ever to penalize a contractor, is wrong; because he, like everybody else in this world, should bear the burden resulting from his own carelessness, negligence, or incompetence. The owner surely has some rights—and this is one of them. A liberal limit of total cost which can be increased or reduced properly, in order to provide for an increase or decrease in the estimated total quantities of materials, will prevent the owner from being excessively overcharged, and still will give the contractor every opportunity to come out whole in any case except that of extraordinary hard luck.

Method *C* is wholly objectionable to the client, in that it places him entirely at the mercy of the contractor and his men. The allowanc

for overhead may be either an assumed percentage, or the actually computed amount in complete detail, the former being generally the less objectionable. Even if the contractor is perfectly honest and has the best will in the world to keep down the cost for the benefit of the client, his employees will not have that desire. In effect, they say to themselves and to each other, "What is the use in my exerting myself unduly? The more the work costs, the more money the 'old man' makes." The author knows this to be the case, for some years ago he had to let a large contract for foreign work at cost plus a percentage; and although the contractors themselves tried to do the honest thing at all times, their men "loafed" to such an extent that the final cost of the construction was atrociously high; and he had occasionally, on his own responsibility, to discharge some of the contractor's employees, including once the field superintendent. This method of letting work involves asking too much of frail human nature.

Method *D* involves less labor in cost-keeping than Method *C*; but, otherwise, it is open to the same general objection.

Method *E* is almost as unsatisfactory to the client as Methods *C* and *D*, except that the contractor's reward for his own iniquity is a fixed quantity and not in direct proportion to the extent of that iniquity.

Method *F* involves a slight improvement on Method *E*, but only to the extent of a little simplification in bookkeeping and perhaps a reduced opportunity for "squeezing" the client.

The author readily acknowledges that during war times, when the trend of the market for both materials and labor was rapidly upward, no contractor could have afforded to take work either for a lump sum or by unit prices. Unfortunately, in order to bring the war to a successful conclusion, a vast amount of public work had to be done with the utmost despatch, irrespective of what the cost might be; hence the Government had no choice at all in the matter, and, consequently, it let many millions of dollars, worth of contracts at "cost plus a percentage" or "cost plus a lump sum." If the true history of all such contracts was ever written and made public, the nation would stand aghast at the extravagance they involved; and those two methods of contract-letting would receive the universal condemnation of all intelligent, disinterested persons.

As was stated in the oral discussion of Mr. Clarke's paper, when a contractor has simultaneously two or more contracts, one of which is on the "cost plus" basis and the other or others on either the "lump-sum" or the "unit-price" basis, he will naturally put his best and most energetic men on the latter, and will shift the lazy and incompetent ones to the former. This practice has become so well established by custom that the "cost plus" contracts have been dubbed "hospital jobs"; and it appears that the nickname has stuck.

Is it not obvious that anyone who lets a contract on the "cost plus" basis places himself absolutely at the mercy of the contractor and the contractor's employees? It is true that the specifications often contain

restrictions which tend to lessen the contractor's power to take advantage of the client; but their enforcement would be very troublesome, and would generally involve litigation with its attendant delay and expense.

Most people will acknowledge that the percentage of truly conscientious contractors is not overwhelmingly large, but how much smaller is that of truly conscientious workmen! The author does not deny that there are workmen who always give a *quid pro quo* and who are upright and honorable in all their dealings; but, alas, they are sadly in the minority. Their number is so small that they are unable to induce their co-laborers to exert themselves any more than they are compelled to, unless they are paid by the job instead of by the day or hour.

By the way, when it is practicable, such a scheme of paying the workmen is an improvement on that of time compensation, because it provides a great incentive to effort; but, at the same time, it also serves as a strong temptation to scamp the work. With close supervision, however, and a strict enforcement of the clause in the specifications relating to the taking out and replacing of defectively built work, the employees soon learn, through the fines and penalties enforced by the contractor, that scamping does not pay, and that the old adage of honesty being the best policy is just as applicable now as it was when first enunciated.

Method *G* involves only a very slight modification of that of "cost plus a lump sum," the said lump sum being replaced by another sum obtained by adding together the products of the actual quantities of all the materials by certain small unit prices agreed on in the contract. Although the author concedes that this method is undoubtedly the best of all the straight "cost-plus" methods, it possesses all the serious objections inherent thereto.

In addition to those previously indicated, there might be mentioned the fact that any straight "cost-plus" basis effectively cuts out competition, and advantages a favored few of the larger and more experienced contractors, rendering it difficult for the smaller and less experienced ones to secure any work, except through some other method of letting. It ought to be evident to anyone possessed of ordinary vision that such a method will militate toward cutting the "small fry" contractors out of bidding; for when an owner is willing to let a piece of work on any straight "cost-plus" basis, he naturally will want to award it to a large contractor of means, who has an established reputation for fairness and efficiency. That would practically mean letting all contract work without competition, and American contractors, as a body, would object seriously to any such procedure. It is true that the owner might call for competitive bids on the basis of having each bidder name a lump-sum as a fixed net fee, and awarding the contract to the competitor who quotes the lowest figure; but the adoption of such a method would often result in serious trouble, delay, and expense, and would not ensure that the work would go to the most desirable bidder.

Under the heading H, a number of profit-sharing methods have been tried and have proved to be more or less satisfactory. All of them necessarily presuppose a careful accounting of cost from start to finish. Unless the contract between the two parties clearly indicates how every main detail of cost is to be computed, there will be trouble before the work is finished. For instance, in respect to plant—does the contractor furnish it free of charge, or does he receive rental for it, with the rental charged as one of the items of cost of the work? How about paying for repairs and renewals to plant? Who stands the expense of these items—does the contractor, or is it charged as an item of cost of doing the work? Again, if extra work is done under the contract, how is it to be counted when making the final settlement?

It requires the service of an expert consulting engineer or that of an experienced contractor to draft a contract and specifications which will provide, in a manner satisfactory to both parties, for all possible contingencies. With such papers, however, and with a close system of cost-accounting, this general method of profit-sharing is the most satisfactory scheme for contract-letting which can be evolved.

None of the modified methods of the "cost-plus" system, involving some means or other of profit-sharing, which have yet been tried in practice, can be said to be entirely satisfactory to the owner, though possibly so to the contractor, in that they all fail to put a limit on the total cost of the construction or to penalize a contractor who, through either wilfulness or carelessness, allows the cost of construction to pass the bounds of reason. It does not suffice to stipulate in the contract that, when the total cost passes a certain amount, the allowance for profit is gradually to be reduced until a certain minimum limit, however small it may be, is reached. The setting of that limit leaves the contractor in a position to take life easily and to avoid personal worry after his hard luck has attained to a certain magnitude; for, subsequently to that, he will lose nothing but his time and the possible use of his plant on some remunerative contract, while the owner will have to pay whatever additional amount the job may cost. On this point the writer knows whereof he speaks; because one of the war-time contracts engineered by his firm was, of necessity, let on that basis, and the results thereof are simply sickening. The client was left at the mercy of the contractor, and the total cost proved to be excessive.

From the preceding it is evident that the "cost-plus," the "lump sum," and the "unit-price" methods of letting contracts are not only faulty, but also unjust to one or other of the two parties to the agreement; consequently, the question arises: "Is there not some method which will be just and fair to both? That question, the writer claims, can truly be answered in the affirmative; but before proceeding to explain such a method in complete detail there will be presented a statement of the main requirements of an ideal system.

Salient Features of an Ideal System of Contract-Letting and Profit-Shar-

ing. The essential requirements of an ideal type of contract are as follows:

First. It must provide a means of sharing with the workmen on an equitable basis the total net profit on the job.

Second. It must set some kind of a limit to the total cost of the work, so as to prevent a careless, incompetent, or conscienceless contractor from running up the expense to an outrageously great amount.

Third. It must reduce to a minimum the chance of the contractor's being out of pocket on the completion of the work, unless such condition is due to his own carelessness or lack of push.

Fourth. It must retain all the advantages of competitive bidding, so as to give every capable and worthy contractor who is desirous of figuring on the work an even chance of securing the contract.

Fifth. It must provide an incentive for the contractor and all his assistants and workmen to use every legitimate effort to make the work as inexpensive as possible, without violating in any manner the requirements of the specifications.

Sixth. It must provide a just and equitable basis of payment for a possible increase in the estimate of total quantities and for adjusting satisfactorily to all concerned the reduction of payment due to a possible diminution thereof.

Seventh. It must ensure that the owner will be acting for his own best interests by aiding the contractor in every possible way to complete his work quickly and inexpensively, provided, of course, that it is done in such a manner as to guarantee the attainment of the owner's ultimate purpose, as expressed in the specifications.

Eighth. Its provisions must be such as to keep constantly in good humor every one connected with the construction.

Ninth. Its method of final settlement of accounts must be clear, simple, and easy of application; and the keeping of them during the progress of the work must be no more complicated or expensive than it would be in the case of any ordinary "cost-plus" contract.

Description of the Ideal Method. Let the specifications, which should invariably be drafted by an engineer who is acknowledged to be an expert in the class of work covered in the proposed contract, be complete and thorough in every detail, recording all that is known concerning the governing conditions; pointing out all features about which there is any uncertainty; tabulating as accurately as possible the estimated quantities of all the materials that will probably enter the construction; providing a justly-drawn clause for unclassified work and the payment therefor; calling for each bidder to submit in full detail his estimate of actual cost of doing the work by applying to all quantities of materials given in the specifications, unit-cost prices (termed Schedule A), each price containing a proportionate share of any contingency allowance that may have been made, totaling the products so as to form "Sum A," and adding thereto the amount of profit

which he has decided to ask, thus making "Sum B." This last amount will, in reality, represent the bidder's tender; but to it there will be added a profit for the owner, exactly equal to that asked for by the successful bidder, and another profit or bonus for the employees, amounting to a previously fixed percentage (say, 20 or 25) of the sum of the aforesaid profits of the contractor and the owner, thus making "Sum C." This last sum is the temporary limit of total expenditure on the part of the owner, predicated upon the assumption that the approximate quantities of materials given in the specifications are correct; and it is on the basis of these "Sums C" that bids will be compared and the award of the contract made. The ratio r of "Sum C" to "Sum A" is to be applied to each of the unit prices used in the preparation of the cost estimate, in order to obtain the list of unit prices (termed "Schedule B") to apply temporarily to the actual quantities of materials in the completed construction, when making the final adjustment of accounts.

If there are any items of expense of construction not covered by the list given in the specifications, the clause of the latter relating to "Unclassified Work" will take care of them. That clause should stipulate that, for all such unlisted items, the actual cost of labor and materials therefor, without any allowance for superintendence or overhead, is to be recorded; and to it is to be added later 30% of its amount to allow for superintendence, overhead, and the various profits. This sum is to be added to the total value of the actual quantities of all the materials listed in the specifications figured at the proportionately increased unit prices as given in "Schedule B"; and the result, "Sum D" (with a single modification explained hereinafter), will be the final limiting cost to the owner and the basis for computing the net profits to be divided between the contractor, the owner, and the workmen.

The specifications, of course, will contain a clause providing a surety company bond for the faithful performance of the work and for guaranteeing the client against having to pay more than the limiting sum agreed on (as finally modified).

Method of Profit-Sharing Contract. The following method of profit-sharing between the contractor, the owner, and the employees is to be adopted:

An accurate estimate of cost of every detail of the work from start to finish is to be kept by the contractor and verified by an accountant in the employ of the client, so that the total profit on the job may be ascertained by deducting this total cost from the maximum figure named in the contractor's tender and afterward embodied in the contract (modified, however, as hereinafter described). This profit, less the amount of the employees' bonus, is to be shared between the contractor and the client as indicated in the profit diagram, Fig. 33*a**. It should be clearly understood that every

* Mr. Hardesty has pointed out the fact that on large contracts the curves cannot be read with sufficient accuracy for a proper final settlement of the account, and that much trouble might be engendered thereby between the accountants of the two parties.

direct and indirect expense to which the contractor is put in doing the work, after the contract is signed, is to be included in the cost—all overhead expenses of every kind, plant deterioration, traveling expenses, supervision, and salaries, excepting only that the contractor himself is not entitled to any salary. In the case of a firm being the contractor, the head of that firm should receive no salary; but if any of the juniors devote their time exclusively to the job, it would be legitimate to allow them reasonable salaries, equivalent to what would have to be paid to regular assistants. All such matters, of course, should be stipulated in the contract.

In order to determine, after the entire job is finished, the amount due the contractor, "Sum C" is to be subtracted from "Sum D," and the ratio which this difference (either a positive or a negative quantity) bears to

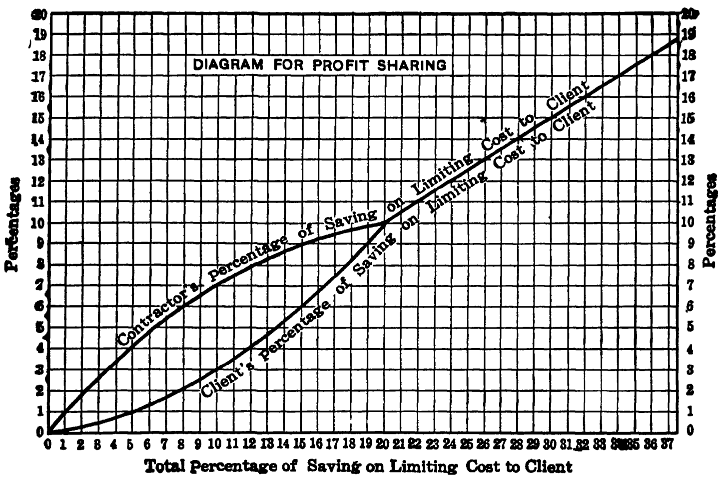

FIG. 33a. Total Percentage of Saving on Limiting Cost to Client.

"Sum C" is to be figured and adopted in the use of the diagram of "corrective ratios" (Fig. 33b*) for the said difference.

Application of Corrective Ratio. There are two reasons for applying this corrective ratio:

First. In the case where the actual quantities of materials exceed the

The solution of the difficulty is to substitute a broken line for each curve, passing through the known points at zero, five, ten, fifteen, and twenty percentages. The readings for these points on the upper line are zero, four, seven, nine, and ten; and on the lower line they are zero, one, three, six, and ten. Such an arrangement removes every possibility of dispute concerning the reading of the diagram, because all percentages intermediate to the above-mentioned ones can be directly interpolated.

* The curve in this diagram is intended to be the quadrant of a circle; hence, if in reading it there be any dispute between the accountants of the two parties, it can be settled by drawing the curve on cross-section paper using a large scale, or by employing exact mathematical formulæ.

estimated ones of the specifications, it would be hardly fair to the owner to apply to the excess those unit prices which produce his tentative limiting expenditure.

Second. In the case where the actual quantities of materials are less than the estimated ones, it would be unjust to the contractor to use the high unit prices on the diminution quantities, not only because of the great difference between these and the unit actual costs, but, also, for the reason that the total overhead charges would be about the same for the diminished amounts as for the estimated total quantities.

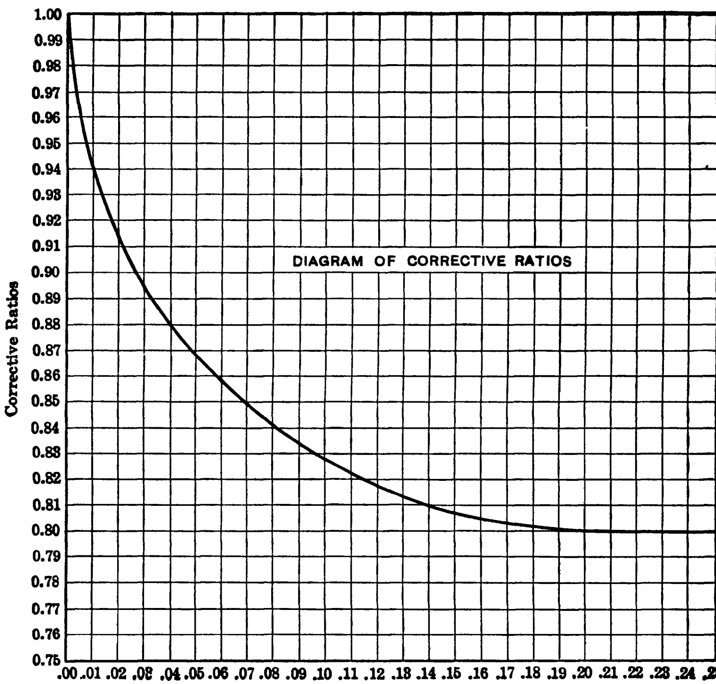

FIG. 33*b*. Diagram of Corrective Ratios.

In the corrective ratio diagram (Fig. 33*b*) it will be noticed, that, after the ratio of value difference (due to increase or diminution of quantities of materials) reaches 0.2, the "corrective ratio" remains constant at 0.8, which corresponds approximately to actual cost conditions. The object of this is to provide that the contractor shall not be too much benefited by an abnormal increase in quantities, nor, on the other hand, shall he be at too much disadvantage because of an abnormal diminution thereof.

To utilize the corrective ratio diagram (Fig. 33*b*) look on the line of abscissæ for the ratio of cost difference, pass vertically upward to the curve (or right line, as the case may be), then horizontally to the extreme

left vertical, which will indicate the corrective ratio required. Next, multiply the previously computed difference by this corrective ratio, and add the result to or subtract it from "Sum C." The result, "Sum E," will be the finally corrected limit, from which must be subtracted the total actual cost so as to determine the amount of profit to be divided. The first step in such division is to set aside the employees' share on the basis of percentage agreed on; and the next is to divide the remaining profit between the contractor and the owner, as per the profit diagram (Fig. 33*a*).

The size of the percentage of the declared profit to allow the employees will depend on the character of the work covered in the contract in respect to the proportionate division of the cost between materials and labor. Under ordinary conditions, the division is about half and half, in which case the employees' percentage should be from 20 to 25; but where the labor cost preponderates these figures should be increased, and where the materials cost is the greater they should be diminished.

In respect to the division of this bonus among the employees, the following method is suggested:

Only those workmen or assistants of any class who have stayed on the job, either until its completion or until their services were no longer needed, should participate in the profits; and the amount of the share of each such workman and assistant should be in the proportion which his total earning on the work bears to the grand total of the earnings of all those employees who so participate. As, in any good business organization, a record is always kept of the amount of salary or wages paid to each employee on any contract, it would require only a few hours of extra work for the bookkeeper, after the job is finished, to compute each man's proportionate share of the bonus.

There is an additional protection against possible loss which might be given to the contractor under certain conditions, especially on work to be done in a foreign country. If it be anticipated that during construction any large general rise in the price of labor is likely to occur, thus greatly augmenting the total cost of the work, the limiting total expenditure of the owner, as hereinbefore finally adjusted, should be increased by an amount figured thus:

Determine the average current wages for common labor at the time of letting the contract, and the average paid therefor during the entire time occupied by the construction, and call the ratio of these two averages r'; then figure the total amount of salaries and wages paid to employees from start to finish and call it W. Then,

$$W(r'-1)\times 1.2$$

will be the amount required, the factor, 1.2, covering a fair allowance for overhead and profits.

It has been suggested of late in the technical press that the contractor himself should be paid a salary by the owner, in addition to whatever

profit he may make on the job. Such a policy would be simply suicidal on the part of the owner, for if the work were to be handled badly, the contractor might continue to earn money while the owner would be losing heavily. The writer has met with just such a case; hence, in this particular, he certainly knows whereof he speaks. Such a practice should never, under any conditions, be countenanced by either the owner or his engineer.

Exemplification. In order to illustrate the *modus operandi* of this method of profit-sharing, let us assume the following case, in which the estimated quantities are exceeded. For the purpose of simplification in figuring, round numbers have been assumed for both the quantities of materials and the unit costs thereof. The job is one of railroad construction and the number of items is intentionally limited for the sake of convenience.

The following are the quantities of materials supposed to be stated in the specifications:

Earthwork, measured in cutting.........	1,000,000 cu. yds.
Loose rock, " " "	100,000 "
Solid rock, " " "	40,000 "
Concrete in structures..................	10,000 "
Wooden trestle.........................	2,000 lin. ft.
Structural steelwork, erected............	500,000 lbs.

The tender of the successful bidder was as follows:

	Quantities.		Schedule A.		Totals.
Earthwork..................	1,000,000 cu. yds.	@	$0.50	=	$500,000
Loose rock..................	100,000 "	@	1.00	=	100,000
Solid rock..................	40,000 "	@	1.50	=	60,000
Concrete..................	10,000 "	@	20.00	=	200,000
Wooden trestle.............	2,000 lin. ft.	@	50.00	=	100,000
Steelwork..................	500,000 lbs.	@	0.08	=	40,000

Total estimated cost ("Sum A")..................	=	$1,000,000
Profit required, 10%..................................	=	100,000
Tender ("Sum B").............................	=	$1,100,000
Allowance for owner's profit............................	=	100,000
Employees' profit, 25% of $200,000.....................	=	50,000
Temporary limit ("Sum C")......................	=	$1,250,000

$$\text{Ratio, } r=\frac{\text{Sum } C}{\text{Sum } A}=\frac{\$1,250,000}{\$1,000,000}=1.25.$$

The proportionately increased unit prices, therefore, will be as follows:

Schedule B.

Earthwork	$0.50×1.25=$0.625
Loose rock	1.00×1.25= 1.25
Solid rock	1.50×1.25= 1.875
Concrete	20.00×1.25=25.00
Wooden trestle	50.00×1.25=62.50
Steelwork	0.08×1.25= 0.10

The actual quantities of the materials in the completed job were, as follows:

Earthwork	980,000 cu. yds.
Loose rock	110,000 "
Solid rock	50,000 "
Concrete	10,500 "
Wooden trestle	2,100 lin. ft.
Steelwork	480,000 lbs.

In addition, there was done by the contractor certain "unclassified work" which actually cost him for labor and materials $20,000.

The revised estimate is, therefore, as follows:

Earthwork	980,000 cu. yds.	@	$0.625 =	$612,500
Loose rock	110,000 "	@	1.25 =	137,500
Solid rock	50,000 "	@	1.875 =	93,750
Concrete	10,500 "	@	25.00 =	262,500
Wooden trestle	2,100 lin. ft.	@	62.50 =	131,250
Steelwork	480,000 lbs.	@	0.10 =	48,000
Unclassified work			$20,000×1.3 =	26,000
Summation, or "Sum D"				=$1,311,500

The difference between "Sum D" and "Sum C" equals

$$\$1,311,500-\$1,250,000=\$61,500.$$

$$\text{Ratio of difference}=\frac{\$61,500}{\$1,250,000}=0.0492;\ \text{say, } 0.05.$$

For this ratio, the diagram, Fig. 33*b*, gives a corrective ratio of 0.865, which multiplied by $61,500 gives $53,198, say, $53,200, making the corrected limit, or "Sum E," = $1,250,000 + $53,200 = $1,303,200.

If the total actual cost of the work, including that of the unclassified work without allowance for superintendence or overhead, amounted to $1,100,000, the total profit would be:

$$\$1,303,200-\$1,100,000=\$203,200.$$

Of this, the joint share of the contractor and the owner would be:

$$\$203,200\div 1.25=\$162,560;$$

and the employees' bonus would be:

$$\$203{,}200-\$162{,}560=\$40{,}640.$$

which is exactly 25% of $162,560.

In respect to the division of this last amount between the contractor and the owner, its ratio to total cost is:

$$\$162{,}560\div\$1{,}100{,}000=0.148, \text{ or } 14.8\%.$$

From the diagram for profit division (Fig. 33*a*) we find the division of this percentage to be:

Contractor	8.9%
Owner	5.9%

This makes the total payment by the owner to the contractor:

$$\$1{,}100{,}000\times 108.9=\$1{,}197{,}900.$$

Let us now take a case where there is a diminution in the estimated quantities of materials. Using the same case as before in respect to estimated quantities and tender, we shall assume the following actual quantities of materials:

Earthwork	1,050,000 cu. yds.
Loose rock	50,000 "
Solid rock	20,000 "
Concrete	8,000 "
Wooden trestle	1,800 lin. ft.
Steel work	450,000 lb.

The cost of the unclassified work was $20,000, as in the preceding case. The revised estimate is, therefore, as follows:

Earthwork	1,050,000 cu. yds.	@	$0.625 =	$656,250
Loose rock	50,000 "	@	1.25 =	62,500
Solid rock	20,000 "	@	1.875 =	37,500
Concrete	8,000 "	@	25.00 =	200,000
Wooden trestle	1,800 lin. ft.	@	62.50 =	112,500
Steelwork	450,000 lb.	@	0.10 =	45,000
Unclassified work			$20,000×1.3 =	26,000
Summation, or "Sum D"				=$1,139,750

The difference between "Sum D" and "Sum C" equals

$$\$1{,}139{,}750-\$1{,}250{,}000=-\$110{,}250.$$

$$\text{Ratio of difference}=\frac{\$110{,}250}{\$1{,}250{,}000}=0.0882.$$

For this ratio, the diagram gives a corrective ratio of 0.835, which, multiplied by $110,250, gives $92,059, say, $92,100, making the corrected limit.

"Sum E" = $1,250,000 − $92,100 = $1,157,900.

If the total actual cost of the construction, including that of the unclassified work without allowance for superintendence or overhead, amounted to $880,000, the total profit would be:

$1,157,900 − $880,000 = $277,900.

Of this, the joint share of the contractor and the owner would be:

$277,900 ÷ 1.25 = $222,320,

and the employees' bonus would be:

$277,900 − $222,320 = $55,580,

which is exactly 25% of $222,320.

In respect to the division of this last amount between the contractor and the owner, its ratio to total cost is:

$222,320 ÷ $880,000 = 0.253, or 25.3%.

From the diagram for profit division, (Fig. 33*a*), we find the division of this percentage to be on the "fifty-fifty" basis, making the total payment by the owner to the contractor = $880,000 × 1.1265 = $991,320.

Advantages. The advantages of this method of contract-letting are as follows:

First. While it is true that the client at the outset does not know exactly what the work is going to cost him, he is positive that it will not cost him materially more than a certain amount, provided his engineer's estimate of quantities is about right, as, generally speaking, it certainly ought to be.

Second. The client has the satisfaction of feeling that even if, in his opinion, the limit determined by the contractor's bid is excessive, and that the final net profit on the job, in consequence, will be too large, the said net profit will be shared between them on a "fifty-fifty" basis.

Third. While the client is bound to pay a certain percentage of the joint profit as a bonus to the contractor's employees, generally he will not be out of pocket thereby, but, on the contrary, he will gain; because the incentive that the prospective bonus gives to all hands to labor energetically will save in the total cost much more than the amount of the bonus.

Fourth. All the advantages of competitive bidding are retained by this method, because the fully-capable competitor who tenders the lowest amount for "Sum C" should be awarded the contract. All bids will be on exactly the same basis, no modification of the stipulated method of tendering being permitted. It is understood, of course, that the contract will not

be awarded to any competitor who does not possess the necessary experience, capital, and plant, and who has not an established reputation for doing good and satisfactory work.

Fifth. The contractor, if he was not too keen in bidding, knows that there is almost no chance whatsoever of his losing money on the job; because before doing so he would have to use up his allowance for contingencies, his own estimated profit, a profit of like amount allowed for the owner, and a substantial sum representing the employees' bonus. If he ever does use up all these safeguards, the chances are many to one that the fault therefor is entirely his own, being due to his negligence, lack of forethought, or deficiency in energy and push; and in that event he certainly would deserve to be penalized.

Sixth. All the workmen and salaried employees of the contractor will be satisfied with their job because of the excellent opportunity offered for extra compensation; and they will, of their own accord, work diligently, and occasionally even overtime, in order to expedite the construction. Of their own accord, too, they will run off the job any employee who is a chronic shirker, and they will make it their business to keep everybody busy; because the more cheaply the construction is done the greater will be the bonus to divide among the faithful employees who stick by the work to the finish.

Seventh. The method of profit-sharing given by the diagram for profit division (Fig. 33*a*) is eminently equitable, in that when the net amount is small, nearly all of it goes to the contractor, and, as it augments, a continually increasing proportion of it goes to the owner, up to the point where the total joint profit amounts to 20%, after which the partition is on a "fifty-fifty" basis.

It will be seen that for a total net joint profit of less than 20%, the following divisions will prevail:

With 5% net, 4%	goes	to	the	contractor	and	1%	to	the owner
With 10% net, 7%	"	"	"	"	"	3%	"	" "
With 15% net, 9%	"	"	"	"	"	6%	"	" "

The 20% point was selected for equal division as being the one above which a contract is generally deemed by contractors to be good, slightly below which it is only fair, and much below which it is bad; for it corresponds to a net profit of 10 per cent. That is as small a margin as is generally deemed safe for any bidder to tender upon, and yet it constitutes a satisfactory profit on a finished job. As for limiting the client's share of the profit to one-half—that is reasonable and just, because he would have no moral right to receive more than his partner, the contractor. If the client's share were allowed to increase indefinitely, it is conceivable that, with a very large prospective total profit, the contractor could save money for himself by making the work more expensive.

Any bidder who tenders on the basis of a profit less than 10% should

be looked on askance by the owner and his engineer; and such a competitor should not be awarded the contract, unless he possesses an exceptionally fine reputation for doing good work and for not quitting his job before finishing it. It is true that during very hard times many worthy contractors are willing to work almost for actual cost, in order to keep their workmen employed; and in such cases the good intention should be recognized in making the award. Nevertheless, it nearly always proves to be unsatisfactory to both the owner and his engineer to let a contract for any piece of construction at a figure below its real value.

Eighth. The contractor will feel during the progress of the construction that the client is a partner on the job, and that, therefore, he and his engineers will not be likely to be unnecessarily severe in their requirements, also that they will permit the adoption of all legitimate expense-saving expedients, and will not demand too many frills on the finishing.

Ninth. Owing to the justice and equity involved by this method of contract-letting and profit-sharing, all concerned in the execution of the work will labor whole-heartedly and good-naturedly, avoiding petty squabbles and disagreements; and the result will be earnest, honest effort, a satisfactory piece of construction, and the general contentment of both parties to the agreement.

Tenth. While this method may at first glance have the appearance of being complicated, it is quite simple; and because of the clear manner in which it is explained herein, it is easily utilized in any actual case by following one step at a time the directions given. The nomenclature of "sums" and "schedules" renders the application of the method very easy. Moreover, it must be remembered that it is to be used only once for each contract, and then only after all the work is completed and the accounts are in proper form. Again, the keeping of the accounts is in no way any more complicated than it would be in case any of the "cost-plus" methods were used.

Objections That Have Been Raised to This Method. A few objections, both orally and in print, have been raised to this proposed method of contract-letting and profit-sharing; but they could not have been well considered, for they are not valid. It will be well before closing this discussion to mention them and show wherein they are untenable. They are as follows:

A. It has been asserted that there would be special difficulty in keeping the accounts; but there would be required therefor exactly the same work which would be necessitated by any of the "cost-plus" methods. They all involve a correct record of the amount of every legitimate item of expense to which the contractor is put; and the bookkeeping in any case would certainly require an account with each employee, so as to show how much money had been paid him from start to finish.

B. It has been claimed that the method is too complicated to be useful. On the contrary, as explained previously, it is simple; and the pecul-

iar manner of its presentation herein is such as to render its use merely a matter of following step by step certain clearly written instructions. Moreover, as before indicated, this method of settlement of accounts is used only once, *viz.*, after the completion of the job.

C. It has been stated that uneducated contractors would have difficulty in understanding the method; but the man who made this claim did the average American contractor a grave injustice. The construction contractors in this country are as bright a body of men as one can find anywhere; and certainly they may be trusted to understand anything in reason that affects their interests.

D. It has been claimed that most contractors have a general inherent objection to sharing profits with the owner; but a little consideration will show that there is no sharing of the requested profit until after the estimated cost of the work (under the assumption of unchanged quantities of materials) has been exceeded. It was to clarify this situation that the writer changed his original idea of having each bidder name a lump-sum (corresponding to "Sum C") as a provisional limit of the owner's total expenditure, and a list of unit prices (corresponding to "Schedule B") which, when applied to the estimated quantities of the specifications, would make the sum of the products exactly equal to the said lump-sum, and substituted therefor the method herein described, *viz.*, that of having each bidder submit in detail his estimate of cost and his desired profit, and arranging the method of determining the limiting cost to the owner by two additions to the bidder's tender. This change is simply a concession to prejudice, and does not modify the method proposed by the writer in his letter published in *Contracting* in its issue of September 15th, 1919. The only fundamental change between that presentation of the matter and this one is the inclusion herein of a bonus for the employees.

E. It has been claimed that this method would tend to deceive the owner. He would certainly be stupid if he could not see clearly how its tendency is to cut down the amount that he will pay for the entire work, and that it will set a just limit beyond which, under the worst possible conditions, he cannot be compelled to pay any more.

F. It has been stated, as a reason for favoring the "cost-plus" system, that bonding companies favor it and oppose all other methods of contract-letting. Naturally, they would do so; because, with the "cost-plus" method, their obligation reduces almost to zero, the risk being placed solely on the owner. In truth, with that method adopted, there does not appear to be any valid reason for having a bond at all. When the contractor runs no risk of loss whatsoever, why a surety company bond? Possibly, if the surety companies would consider it from this point of view, they would not oppose the writer's suggested method of contract-letting; for, while it reduces very greatly the possibility of loss to the surety company, it does not destroy that organization's function by removing entirely its *raison d'être.*

G. The claim has been made that this method is not applicable to cases where no estimate of quantities has been made or for which no specifications have been prepared, and when it is necessary to let the work without delay. The correctness of this claim is granted, but its applicability should be confined to war work, in which the element of time is the principal consideration. In civil life, if a projector of an enterprise is in such haste to start his work that he has to omit the preparation of specifications and estimates of quantities before letting the contract, he certainly deserves what is coming to him in case it proves that he has placed himself in the hands of an unscrupulous contractor on the "cost-plus" basis.

H. It has been pointed out that the law will prevent certain public bodies from letting contracts on a "cost-plus" basis, and that such a ruling might apply to the writer's proposed method also. If such really is the case, the remedy would evidently be to modify the law so as to permit of its adoption; but would not the fact of its containing a clause which sets a limit to the contractor's total compensation always render the method legal? This, however, is a question for the lawyers to settle.

I. A prominent writer on economics has lately issued a tirade against gambling, especially as it applies to contractors guaranteeing owners against the cost of work exceeding the amount of the tender. The writer concurs in this theory to the extent that there should be no gambling on the amounts of assumed quantities of materials, or, in certain cases, on there not being any great rise in the average price of labor, or on possible loss through any reasonably great amount of hard luck; but there is certainly nothing unmoral or oppressive in insisting on his gambling to the extent of guaranteeing against his own incompetency, carelessness, or lack of forethought. The writer is of the opinion that a margin against actual money loss to the contractor, consisting of his estimated amount for contingencies, plus his requested profit, plus an equal profit for the owner, plus a substantial bonus for the employees, is amply large to guarantee him against all loss through ordinarily unfavorable eventualities; and that, in ninety-nine cases out of a hundred, if such a margin were exceeded, it would be because of the contractor's own fault.

Adoption of Method. If this proposed method of contract-letting and profit sharing is received with favor by engineers, architects, contractors, and builders in general, it could easily be adopted as a standard for the country by calling a small convention with a single representative from each of the leading technical and railroad societies, contracting organizations, bankers' associations, and labor guilds, to discuss the advisability of adopting it (or else some modification of it) and to report the decision of the meeting to the said bodies for their approval. If any large group of clients, such as the railroad companies, were to adopt the method as standard and use it, very soon everybody having construction contracts to let would follow their example, thus making it the universal standard of contract-letting for the country—nor would it be long before other American coun-

tries would follow our lead, thus greatly simplifying our business relations with the various American Commonwealths.

Addendum. In all lines of manufacture the employees should share in the net annual profits of the company; but, in figuring the yearly cost of running the establishment and doing the work, there should be included fair salaries for all the working officers, 6% on the actual amount of cash invested in the plant and business (but not on the total capital stock), an annual allowance for a sinking fund to redeem the bonds or other indebtedness of the organization, and taxes of every kind. The net profit estimated in this way should generally be divided on the basis of one-third to the employees and two-thirds to the company, but sometimes, perhaps, on that of one-fourth and three-fourths. The reason why it should not be split on a "fifty-fifty" basis is that the company has to run the risk of standing a loss in bad years, while the employees do not. Only those employees who are still connected with the company at the end of the year, or who have been discharged in good standing, should share in the bonus; and their proportionate amounts should be computed as previously indicated for the case of construction contracts.

CHAPTER XXXIV

ECONOMICS OF BRIDGE-ENGINEERING OFFICEWORK

CONCERNING economics in the management of a bridge engineer's office, it will suffice to offer a few general principles and refer the reader to Chapter LVIII of "Bridge Engineering." It has been claimed by some engineers that the scheme of management therein expounded is far too elaborate and costly, some going so far as to state that, if it were followed out exactly, the expense involved would eat up all the profits. Such, though, is not the case, for while it is true that it is too expensive for an office with a small force, it is not so for one handling simultaneously many millions of dollars' worth of bridgework, as did the author's in *ante-bellum* days. It represents an ideal system worked out with great care and in complete detail; and if it were utilized with proper discretion by bridge engineers, bearing in mind that one should "cut his coat according to his cloth," much benefit would result.

In any case, though, the following principles should be observed:

First. All employees should arrive promptly in the morning, preferably a few minutes ahead of time, should get to work immediately, should work diligently, and should put in full time. If, for any unavoidable reason, an employee loses some time from his work, it should be a point of honor with him to make it up by working overtime.

Second. Talking among the employees during office hours should be reduced to an absolute minimum consistent with a proper exchange of ideas as to the development of the work of designing and detailing. No general conversation in the office should be allowed under any circumstances whatsoever.

Third. No visitors should be permitted to enter the drafting or computing rooms, and callers upon employees should be made to understand that they are not welcome and that visiting is against the rules.

Fourth. No smoking should be allowed during office hours. It takes valuable time from the work; and hot cigarette ashes are very destructive to tracings.

Fifth. Each employee should be made to attend strictly to his own business, and no one should be allowed to pry into matters in relation to which he has no legitimate concern.

Sixth. Before any computations on a design are made, full data should be collected therefor; and a complete list of the conditions precedent should be sent to the client for approval in writing before any serious work is done.

Then, if later any changes are called for by the client, they would be made at his expense, and he could raise no valid objection to standing the cost thereof.

In order to systematize the collection of data, each office should have a printed list of questions or memoranda to send to clients, agents, or field men; and these should be filled in as fully as practicable for each job. Such a list is given in Chapter XLVI of "Bridge Engineering," but lately the author, for the benefit of his future practice (especially in foreign countries) has materially elaborated this. He feels that it cannot well be made too full or complete, because the more one knows in advance about the governing conditions the better will he make his design.

Seventh. There should be established certain limits to the accuracy of all calculations, and these should be adhered to. The list of limits adopted in the author's practice is given on page 1377 of "Bridge Engineering."

Eighth. After each page of calculations is finished, it should be checked by the same computer so that, if he has made any error thereon, it may be corrected; and thus its effect will not be carried into any succeeding pages

Ninth. All results should be roughly checked by the computer, using old records or diagrams, so as to ensure that no egregious blunder has been made.

Tenth. All calculations should be checked by an independent computer before being turned over to the drafting room.

Eleventh. Every record, book, pamphlet, and similar office possession should be filed and indexed so that it can be found at any time without delay.

Twelfth. Enough drawings should be made to enable the contractor to prepare properly and readily his shop drawings or other working drawings—but no more; and the preparation of shop drawings in the engineer's office should be strictly avoided. If they are made there so as to suit the style of one shop, they would probably not satisfy the idiosyncrasies of another—hence it is better to let each shop prepare its own shop drawings.

Thirteenth. It is truly economic to use standard parts whenever this is practicable. It saves time in the office and money in manufacture. Special sections of metal should be avoided, even if they apparently be economical; for generally time is far more valuable than a little extra metal. It is only in case of a large amount of duplication that special sections are legitimate.

Fourteenth. A simple style of lettering is both neat and economic, and the use of stencils and the printing press involves the saving of time and money.

Fifteenth. No unchecked drawing should ever be allowed to go out of the office—no matter how pressing the call for it may be. The method of checking all drawings should be thorough and systematic, and it should invariably be followed.

Sixteenth. When changes on drawings become necessary, they should,

of course, be made; but their effects on all parts should invariably be followed out to the utmost limit of their influence.

Seventeenth. Handling of the office work should be done by a thorough and approved system, such as that given on page 1387 *et seq.* of "Bridge Engineering."

Eighteenth. Proper blank forms facilitate office work and therefore economize on cost.

Nineteenth. Cost records for the various jobs should be kept so as to be able to figure properly on the probable expense that will be involved in doing prospective work.

Twentieth. Card indices should be kept for all possessions as well as for records; and this applies to the engineer's library. No books should be taken from the office without special permission, and a record of all such loans should be kept and utilized for compelling the return of all books thus borrowed.

CHAPTER XXXV

ECONOMICS OF INSPECTION*

The economics of inspection is a subject that is rather intangible, and yet is a branch of economics which really exists and is of great importance; hence it has a proper place in this general study, because it involves a desired result to be accomplished with a minimum expenditure of effort, money, and (to a slight degree) material.

The first point to treat is this important question—To what extent will it pay the owner or promotor to have his work inspected, and how much money should be spent therefor? To this query the answer is clear and unequivocal, viz., that, within the realm of reason, the manufacture and construction of bridgework cannot well receive too thorough inspection, and the owner should be willing to pay for it a reasonable compensation to high-class men. A few dollars saved by employing cheap inspectors may mean very many dollars lost through their blundering and from lowering the value of the finished product.

Like all service functions, there should be a strong distinction between professional service at reasonable rates and simply commercial service rendered on low competitive terms. The quality of inspection is evidently dependent, as is all professional work, upon the character of the men employed, and this is unavoidably dependent upon the compensation allowed.

From the above it will be appreciated that the quality of inspection must, according to the same rule as applies to all business, be in direct proportion to its financial reward. To be of genuine value, inspection must be constant, intelligent, and complete. A final inspection may determine the satisfactory compliance with the contract, but cannot, generally, secure an adequate correction of errors; and certainly it cannot prevent them or tend to the improvement of the work. The criteria of quality of inspection are the experience of the men directly on the job, the time spent on it, and the quality of the final record. The engineer or person having the responsibility of engaging Inspecting Engineers should decide upon the experience and reputation of the firm with which he purposes dealing, should know the

* For a large portion of the data from which this chapter was prepared, the author is indebted to his friend, Mr. Watson Vredenburgh, C. E., of Hildreth & Co., one of the best-known and most successful inspecting bureaus of this country. The first part of the chapter, which is his work, relates to superstructure, while the latter portion from where the treatment of substructure begins represents the author's opinions.

experience of the men to be employed upon the work, and should critically examine the character of both progress and final reports furnished him. He may also properly demand information as to the time of the men employed upon the job.

The method of payment by tons inspected is satisfactory, with a knowledge as to the quality of inspection; but if the Engineer is doubtful as to the character of the work that is to be done, he may arrange his terms on a basis of cost of the actual time of the men employed on the work, plus a percentage or a fixed allowance per ton to the Inspecting Engineers for organization and supervision. It is difficult to fix a proper charge per ton to cover all sizes, kinds, and locations of work which would be economical to the client and fair to the Inspecting Engineer. The latter may properly make a profit from the favorable combination of his work at rolling mills and manufacturing shops, and from the saving of time and traveling expenses, and at the same time, under proper arrangement and knowledge of these conditions, give the client the benefit of any economy arising from these propitious circumstances.

Some Engineers, solely with the false thought of economy in the cost of a structure, omit to specify inspection, or bow to the wish of an owner who may consider the inspection an unnecessary expense, without having any conception of either the details of the service or its benefits. It is not inconceivable that an owner or Engineer who fails to provide for the supervision of manufacture may be held responsible for damage or loss of life resulting from any failure during erection or thereafter. The question may well be asked—What is the use of drawing plans, specifications, and contracts, unless steps are taken to determine that their requirements are being carried out?

Supervision of the manufacture of bridgework may be made by the direct employees of an Engineer or of a Railroad Company; and where this method may be considered, the question of economy as compared with the employment of Inspecting Engineers who make a specialty of such work becomes a factor. The reasons for the existence of the latter class are primarily that the manufacture of structural metalwork is conducted at various rolling mills and at one or more fabricating plants, is in progress at several points at the same time, and is frequently intermittent. If an Engineer uses his own forces for this work, it is essential that a number of men be employed; and there is, consequently, much waste of time and of traveling expenses. To meet this situation, the independent Inspecting Engineer establishes an organization of experienced men who are permanently located at the various manufacturing centers, and, by competent supervision of their work, makes use of their time simultaneously over a number of contracts, thereby tending to efficiency and economy. The overhead expense necessary in the operation and supervision of inspection for any single contract is less with the Inspecting Engineers, as their normal expense of this nature is distributed over a volume of work. The efficiency

of their service can be made at least equal to that of any other inspectors, if the principles of selection referred to previously in this chapter are carefully followed. The Inspection Company, presumably, has a wide knowledge of shop methods and an intimate contact with many shop managers, and from experience is able to handle the defects arising during manufacture with some advantage of practical familiarity, as compared with the Designing Engineer; and the former has personal acquaintance and constant business relations with the shop management which the latter does not possess.

Economy in Performance

The Inspecting Engineers entrusted with the mill and shop inspection of the steel work of any project have directly the solution of the problem of accomplishing the desired result with the least expenditure of effort and money consistent with first-class service.

The systematizing of the service for economical performance involves the selection of inspectors-in-charge having proper experience in the class of work under contract, as well as training in the handling of the relations with the manufacturers, making correct reports, etc. Where the work is of sufficient size and character to warrant more than one inspector at the shop, it can be so arranged that the assistants may be men of lesser experience, with corresponding saving in remuneration, and duties assigned to them accordingly. The assistants can supervise the routine work of assembling, punching, and riveting before completion of the finished members, and cleaning, painting, weighing, and loading before shipment, besides estimating weights and making proper reports, while the chief inspector can be made responsible for the relations with the shop, the planning of the work to meet the field conditions, the delivery of material from the mills, the actual inspection of finished members for measurements and character of workmanship, the shipment of material in the order and manner desired at the building site, and the making of final reports to his headquarters, in accordance with instructions, so that the client can be furnished with a complete descriptive report which will serve as a valuable record of the manufacture.

Under a proper system of organization there are a number of considerations in connection with the method of inspection which should have attention, in order that the service may be economical and successful. The inspectors at both mills and shops should be placed on the work as soon as it starts; their instructions as to specifications and plans should be prompt and complete; their co-operation with the mills should be such as to secure the rolling and shipment of material to the shop, in accordance with the order of manufacture there, so as to prevent errors or discover defects as early as possible in the work, and thereby save time in manufacture and delay in shipment, with the additional advantage of avoiding being compelled to allow the work to be patched by corrections; and the co-operation

should be extended so as to secure the cleaning and painting at the proper time, and under correct conditions, in order to save cost in labor, time, and materials of repainting, either at the shop or in the field. The inspection should cover such co-operation with the management as to secure good work with the least expense to the manufacturer, and the shipment of the finished product at the time and in the order necessary for expeditious and economical erection.

Much that is stated in Chapter II, under the heading "Economics of Mental Effort," is important to apply to the service of inspectors as well as to that of the Designing Engineers. It would be well for the Inspection Firms and the Supervising Inspectors to study and practice the considerations referred to therein, and apply the knowledge gained to the selection and direction of their inspectors actually performing the duties. The inspectors should be selected not only for their experience but for temperament to fit them for the very important duty of co-operation with manufacturers; also with regard to their health and habits. In all cases where inspectors have demonstrated their fitness and loyalty to the employer's interests, the direction of the work should be handled with every consideration for the employee that is consistent with the nature of the work and with other conditions affecting the clients' interests. This relation between Inspecting Engineers and their employees in charge of work affects directly the character of the service and, therefore, the economics of inspection.

The inspection in the field of the placing of foundations, the building of masonry, and the erection of metalwork needs, from an economic viewpoint, the same consideration as does the manufacture of the bridge superstructure. The inspectors should be men of experience; their duties should be so laid out as to promote the progress of the fieldwork, and prevent the rejection and rebuilding of portions of the work; their idle time should be applied to other features of the project when the actual fieldwork does not demand their attention; and they should be subject to the same considerations of the "Economics of Mental Effort."

In general, it can very properly be considered poor economy to have given attention to the economics of promotion and secured competent design and specifications, and not to have provided for inspection to make sure of the entire manufacture and erection being performed in strict compliance with them; or to have had incompetent inspection by persons inexperienced and without proper organization; or to have failed to secure a fair and reasonable compensation for Inspecting Engineers fully qualified by experience and organization to perform the required service in a manner commensurate with its importance.

CHAPTER XXXVI

ECONOMICS OF SHOPWORK

WITHOUT the aid of a thoroughly posted bridge-shop engineer, it would be entirely impracticable for anyone who is not truly experienced in structural-steel manufacture to write at all intelligently upon the economics of that branch of bridgework. The author encountered great difficulty in securing the needed aid, partially because the work involved in the preparation of the notes called for many hours of a busy man's time, but mainly because those who are best posted on such characteristically practical matters are not accustomed to express their thoughts on paper.

Failure so to collect one's knowledge is a serious drawback to any man; for he never can determine wherein that knowledge is hazy or lacking until after he has attempted to collect, correlate, and systematize all that he knows upon the subject at issue. Many engineers and others, who in times past have done the author the honor of supplying him with special information, have afterwards assured him that they felt well repaid for the time and effort which they had devoted to the work, through their increase in knowledge obtained in making the investigation. Many a time and oft in his professional career has the author personally proved the correctness of this principle; and he earnestly recommends its serious consideration to the younger members of the engineering profession.

Fortunately in this case, from his old friend, Mr. Thomas Earle, C.E., Vice-President of the Bethlehem Steel Bridge Corporation, the author succeeded in securing the information of which he was in search; and he feels very thankful for the aid rendered, because it is certainly a great concession and a real favor, in the case of an exceedingly busy man, to take the trouble to collect and systematize the special knowledge which he obtained by many years of hard work. The most of what follows in this chapter is essentially the substance of the data so courteously furnished by Mr. Earle.

The economics of design to meet shop conditions has been discussed at length in Chapter XXIII; and Mr. Canady's contribution thereto covers very thoroughly the ground of economics in the drafting-room of a bridge shop; hence there is left for treatment only the economics of doing the work in the shops themselves, together with certain allied economic subjects.

The general economic problem in shopwork is to attain a certain result with the minimum expenditure of effort, time, and money. Each piece of

material, therefore, must be taken up as few times as possible and carried the shortest practicable distance in securing the desired results. A portion of the problem must be solved before any physical work is performed, because the design of the shop will have a large bearing upon it. The Contracting Department of the organization is also an economic factor of considerable importance in shopwork.

A shop that is designed to handle all classes of fabricated steelwork will not handle any one particular class to the greatest advantage; hence, before the shop layout is made, this matter will have to receive careful consideration. If a general miscellaneous class of fabricated work is expected, the shop should be designed to suit such mixed fabrication; but if the circumstances are such that it is probable that the bulk of the manufacture will be of a particular character, it will be advisable to design the shop so as to handle that type of work to best advantage. If that is done, and if the Contracting Department of the organization keeps the shop filled most of the time with fabrication of a different class, the results will be unsatisfactory, thus proving the *modus operandi* to be uneconomic. It is true that at certain times it is impracticable to secure work of the character best suited to the shop, and then the results will inevitably be uneconomic; but it is better to keep the force occupied and everything moving, even at a disadvantage, rather than either to close down entirely or to let a portion of the men be idle. In that case it might eventually prove truly economical to operate temporarily at a small pecuniary loss. Nothing in shop or office is so disheartening or so disorganizing as idleness of employees; and when there is not enough work on hand to keep everybody on the *qui vive*, the general effort slackens and the efficiency of the entire organization is lowered. For this reason, in bad times it really pays to do work at actual cost or even a trifle below, so that, when the revival of business comes, all in the organization will be ready to tackle the new work with energy and efficiency. If, during the larger part of the time, work of the proper character is going through the shop, and if the results are particularly favorable, this will more than offset the uneconomics due to unsuitable operation for short periods.

The primary economic feature of any shop-layout is the elimination of unnecessary transportation and the provision for rapid passage therethrough of all the materials which are being fabricated. The usual arrangement is to transport the metal longitudinally through on surface tracks and to carry it transversely by traveling cranes, thus reaching expeditiously every portion of the floor space. In some shops the cars are pushed along the tracks by man power, while in others electric or gasoline energy is employed. In these days of almost universal power operation and of high-priced-labor, it certainly is economic to use power for traction purposes. Of course, the cranes are operated electrically. Sometimes they are handled by special operators who ride on them, but often by the workmen on the floor by means of hanging ropes arranged in a very ingenious

manner, so as perfectly to control all the operations of the moving apparatus.

Assuming that the work to be fabricated is miscellaneous bridgework, including movable spans, there are certain factors relating to major operations and numerous others that pertain to specific parts of the work; and all of them, according to their effectiveness, influence more or less the general economics of the shopwork.

Principal Economic Factors

The most important factors covering all operations are light, heat, ventilation, space, handling of work, and management of men. The first four are, of course, dependent upon shop design; but if they did not receive proper consideration at the time the shop was planned, a study should be made in relation thereto; and any resulting proposed changes that promise to provide more economic fabrication should be inaugurated with the least possible delay. A relatively small amount of money saved every day will warrant the expenditure of a considerable sum.

Similarly, it is truly an economic policy to scrap any tool, machine, or apparatus which can be replaced by one of decidedly greater efficiency, even if the one condemned be practically new. Economic results are what should be striven for, regardless of the difficulties and expenses that are inevitably inherent in the making of changes. It is this willingness of the American manufacturer to scrap all apparatus that can advantageously be replaced by something more effective, which has often given him the supremacy over his foreign competitors. It is the truly-up-to-date operator who proves most successful in his business.

Light

A well-lighted shop is necessary to enable the men to read the drawings readily, to decipher the marks on the metal, to find the material expeditiously, and to assemble it properly. Good light makes it possible to handle the metal rapidly, to inspect the work effectively, and to detect errors. It makes the shop safer and more pleasant to work in; and it has a beneficial psychological effect upon the spirits of the men, because, when everything about the place is bright and cheerful, they not only operate to greater advantage but their brains function better, with the ultimate result of a largely increased output.

By painting the interior of the shop throughout in white and keeping the paint comparatively clean through frequent washings and occasional renewal, the visibility of everything which it contains is greatly augmented, both in daylight and after dark, provided that a proper system of electric lighting is installed and maintained. It does not pay to retain old, dim lamps; hence a supply of fresh ones should be constantly on hand and installed whenever necessary. Moreover, powerful lamps should always

be used, because the small amount of electric current saved by employing weak ones is a bagatelle in comparison with the benefits obtained through ample lighting capacity.

Heat

While in the past some bridge shops have been run during the winter months without heat, the furnishing of a reasonable amount of it then will assuredly result in a sufficient increase of tonnage in a given period to pay for several times its cost. No man who is uncomfortably cold when housed can work to advantage, although it is true that out-of-doors laborers in winter often have to keep busy in order to maintain the necessary blood circulation. So much attention has been paid in late years to the conditions under which labor operates that it is probable that at the present time in most of the northern states it would be impossible to induce men to work during the winter months in an unheated shop. It is certain that such a shop would not attract the best character of labor; and efficient workmen are essential to the production of economic results.

Ventilation

In most bridge shops certain operations involve the production of considerable quantities of smoke and gas; and it is essential to arrange for their removal as rapidly as generated. The men cannot do satisfactory work unless they have a sufficient supply of fresh air; and the atmosphere they breathe should be free from gases and odors. Even though these be not actually harmful, their existence militates against economy; because, when such odors are perceptible, the workmen seem to get the impression that all of their ordinary ills and discomforts result therefrom. The importance of ventilation is greater where the design of the shop is such that overhead traveling cranes are operated by men located near the roof, as the heat, smoke, and gases accumulate there and make it impossible for these men to handle their work properly.

Space

The shop should be so designed that all of the work of any particular character can be done in a space by itself; and sufficient area should be provided for handling the maximum output of material for each operation throughout the shop. For instance, if the space devoted to reaming be insufficient, the shop up to that point will become over-crowded with material ready for reaming, the shop beyond will not contain sufficient material, and the tonnage of the work going through will be limited because of the lack of space in this particular area. It is just as important to have enough space for each operation as it is to have sufficient total space. If it be found that there is a lack of space in certain areas, and if more space cannot be secured at these points, the trouble will have to be overcome as

much as possible by placing particularly efficient men in these areas and by making special efforts to prevent their work from being delayed by the other operations. Material must be ready for them as soon as they can handle it, and it must be taken away from them as soon as they have finished with it. This, of course, involves some increase in expense, but the additional tonnage secured in a given time will amply warrant the extra cost.

Handling of Work

The four factors of light, heat, ventilation, and space, all dependent upon shop design, have a direct bearing upon the proper planning of the passage of the work through the shop. A proper planning system is one of the most important factors in shop economics; and certainly it would not work out satisfactorily, unless there were sufficient light, heat, ventilation, and space. Such a system should provide for the location and movement of the material from the time it is delivered at the shop until the time it is shipped therefrom. The Shop Manager should always be able to ascertain immediately in exactly what portion of the shop any particular material is located. Special stress should be laid upon the importance of having ample space for the raw materials as they are received from the mills. Some years ago this was neglected, resulting, in the case of large shops, in unsatisfactory operation, as it was impossible to locate and bring into the shop any particular material on short notice.

There is an ideal feature of economics in shop practice that should be adhered to when it is possible, viz., the continuous passage of all material *in one direction* through the shop. This is not always feasible; but it is the general experience that, whenever the principle is violated by carrying material backward, progress is interfered with, output is lessened, and unit cost of production is increased.

Management of Men

The efficient management of the workmen is one of the most important factors in shop economics; because the maximum of output can only be attained by having each man in the shop labor to his limit of efficiency, all work together harmoniously, and all act in concert under a scientifically-laid-out programme.

If workmen are to develop one hundred per cent of efficiency, or anything like that amount, they must be well fed, properly housed, comfortable in their surroundings, happy, interested in their occupation, contented with their recompense, and amused in their leisure hours. Some of the broad-gauge managers of bridge shops are recognizing these requirements, and are endeavoring to fulfil them in various ways. For instance, cafeterias are being established for the employees, where good, plain, wholesome food is served at actual cost; libraries and reading rooms are being

provided; comfortable cottages with all modern improvements are being built and rented to the men at low rates; playgrounds are being inaugurated; and opportunities for large earnings are being afforded by the adoption of the system of piece-work. If a method of annual profit-sharing like that advocated by the author and expounded in Chapter XXXIII were adopted, in addition to the other welfare methods just mentioned, most of the labor troubles in bridge shops would disappear. Great care should, however, be exercised in giving the employees unusual privileges or rewards, as these may do more harm than good if the employee does not feel that they are due to special effort and that he is receiving them as a payment for work that he has performed. There has recently been too much talk for rights without consideration of duties. Rights result only from the performance of duties. It is most important in the interest of true economy that the amount of each workman's compensation shall be proportionate to the efficiency of his accomplishment. A flat scale of wages is absolutely destructive to progress, output, initiative, and economy.

Safety Considerations

Every effort made by both the management and the employees to increase the safety of the workmen is a step in the line of true economy. Each accident in the shop, no matter how trivial, causes immediately more or less delay, and often incapacitates one or more men for some time. Notwithstanding the fact that such men may be replaced quickly, their successors naturally, for a while at least, are not as efficient; hence progress is impeded by the failure of the new men to function effectively.

Again, many of the injured men return to their work out of condition—some of them permanently so, for instance with the loss of a finger or an eye. Then there must be considered living expenses and doctor's bills during the enforced absence from work. Somebody has to pay these; and while in the end they are borne by society, they are carried primarily by the shop; and that shop which is freest from accidents has the least expense on that account. Other things being equal, it can, therefore, secure the largest percentage of profit. Generally, too, the workman has to be compensated for his injury, which is another economic factor of importance. Possibly, insurance will cover this; but often the company has to pay it.

Much might be said from the humanitarian point of view concerning the prevention of accidents; but this is a treatise on economics, not ethics. Viewed from every possible angle, it is in the line of true economy for the company to take every precaution to prevent accidents in the shop; for safety and efficiency are inseparable. To this end the responsibility should be laid upon one of its officers whose duty it would be to anticipate all possible accidents in the shop and take measures to prevent their occurrence. This could be accomplished in various ways, among which might be mentioned the following:

A. Properly guarding the running parts of all machinery.

B. Frequent inspection of all machines of which the failure could injure anybody.

C. Modification of machinery that is essentially dangerous in any particular.

D. Posting of efficient warnings around the buildings, calling attention to possible dangers.

E. Keeping the floors clean so that they will not become slippery.

F. Prevention of excessive or injurious dust in the atmosphere of the shop.

G. Occasional lectures to the workmen instructing them as to what they should do to avoid accidents.

H. Distribution of pamphlets treating of important matters of safety, sanitation, hygiene, and like subjects.

I. Having available at all times a surgeon to care for anyone who, in spite of all precautions, may be injured.

Smoking*

Smoking in the shop, for several good reasons, is an uneconomic practice and should never be permitted. In the first place, it causes danger from fire; in the second, it occupies a quite-considerable portion of employees' time that should be devoted to work; and in the third, no man's mind can act as keenly when his system is under the soporific influence of nicotine as it can when free from the effects of that narcotic. Very few first-class shops nowadays permit smoking on the premises. The restrictions against smoking apply with even more force in the drafting-room than they do in the shop; for, in addition to the objections just noted, it might pertinently be stated that the dropping of hot ashes from cigarettes onto tracing cloth is not specially conducive to economy. If a man must smoke, let him do it out of working hours and not upon the company's premises, unless there be provided a smoking room for use during the luncheon hour and after one's day's work is done.

Anticipating Troubles

All officials of the company should make it their business to try in every possible manner to anticipate and forestall accidents or occurrences of any kind tending to interfere with shop operations or to reduce efficiency, quality of workmanship, or output. Conferences of officials should be held from time to time for the purpose of discussing this and other matters relating to the general welfare of the works and the workmen.

* Mr. Earle deems the author to be a bit drastic in his views on the uneconomics of smoking.

STANDARDIZATION *

There are few matters of such importance for efficiency and economics as that of standardization; and it should apply not only to tools and product but also to the operations of the workmen. The motion-study expert has proved conclusively that everybody wastes a certain portion of his time and energy in all of his operations; hence it would be economic to make a study of all workmen's motions and indicate to them what they should do in order to correct at least the most glaring of such faults. This has already been tried; and the good results effected have been surprising. Probably the motion-study expert who secures the best results is the foreman who sees that the men learn how to do their work without loss of motion, and then treats them in such a manner as will make them wish to stay where they are.

STOCK MATERIALS

The amount of stock material that is kept on hand is one of the determining factors in the size of space required. It is influenced by the character of the work done, and is dependent upon whether a warehouse supply of material is promptly available. If quick shipments are to be made and high prices obtained for them, a considerable amount of stock material will have to be kept on hand; and it should be stored under shelter so as to be protected from rusting. A day is coming, and it may not be far distant, when the purchaser of a bridge will insist that the structure shall be manufactured of metal which has never been attacked seriously by rust or has

* Concerning this clause and the next succeeding one, Mr. Earle has commented by letter as follows:

I wish to comment somewhat on what you say under the above heading, in connection with what you say under the heading of Stock Materials. The question of Standardization as relating to economy of shopwork is, of course, the smaller portion of the economy of Standardization; for, so long as each engineer wishes his bridges built to his own specification and design, it would be uneconomical for the Shop to store steel under cover. It is, to my mind, absolutely undefensible for standard bridges of standard lengths to be built in accordance with the various specifications, as is now being done. Deck plate-girder spans, through plate-girder spans, and truss spans of two or three types can be built to standard specifications and standard loading and will give absolutely as good service on the New York Central R.R. as they will on the Pennsylvania R.R. or on the Chicago & Northwestern R.R., or any other railroad; and, as far as we can tell at the present time, the life of a bridge under one good specification is the same as the life of a similar bridge under another good specification. If all such bridges were built in accordance with a standard design and specification, it might then be economical for the Bridge Shop to keep material for such structures under cover. The manufacturers would then know that it would be safe to do this; for, if they did not sell to one customer, they could sell to another. Incidentally, this would be a great advantage to customers; because, if they should want a bridge in a great hurry on account of a flood or other accident, it could undoubtedly be secured without delay from a shop having such a bridge in process of manufacture.

been stored or kept for any length of time in any place unprotected from the weather.

Stock material should be ordered to such specifications as will permit of its employment in connection with the usual class of work going through the shop. Each metal section should be kept by itself; and the various lengths thereof should be stored in separate piles. The storage ground should be divided into various areas; and records should be kept of the location of the materials in these areas. It is only in this way that the Contracting Department would be warranted in promising early shipment, in order to secure orders at favorable prices. It is essential for the satisfactory and economical running of the shop that both stock materials and contract materials can be delivered promptly to the machines as soon as it is decided to call for them.

In the case of special material for contracts, just as soon as it is received from the mills it should be stored in the units in which it is later to be brought into the shop, the said units being the metal called for by a bill of material, or shown on a drawing, or indicated by some combination of these methods. A record should be kept of its location, so that, when the templets are completed and the shop is ready to take up the work, it can be delivered in complete units; and it should then be carried through the shop in such units. Too much stress can hardly be laid upon the desirability of bringing in material complete in units, as the lack of any portion thereof would hold up in the shop a large part of the balance of the material of this particular unit; and this material would clog up the shop and either prevent other material from going through or allow it to pass only at additional cost.

Supply of Labor

For several years past the supply of suitable labor for fabricating shops has been limited; and there is no reason to assume that there will soon be any improvement in this respect—in fact the evidence at the present time is to the effect that the difficulties will be aggravated rather than ameliorated; and this condition must receive due thought in considering shop economics. The character of the shop and the way in which the work goes through it have an important bearing upon the securing of a sufficient force of capable employees.

It is of the utmost importance that a permanent force of well-trained workmen be established. Every legitimate effort should be made to retain the services of capable and efficient workmen, because the breaking in of each new man costs, in various ways, considerable money. There is an immense difference between both the quality and the quantity of the daily accomplishment of the trained workman and that of one who is not experienced. As one walks through a bridge shop, even on a short visit, by watching for a few minutes the men at work he can readily distinguish between those who are experienced and those who are not; and when a new

shop is opened with untrained workmen, it takes many months to bring it into any condition at all approaching the ideal.

Tool Equipment

A comparatively large expenditure is warranted for securing labor-saving tools. Labor is a very undependable element; but the tools, if given proper attention, will take care of the work at all times. In the case of shops of equal capacity, the difficulties of running them successfully seem to increase about in proportion to the square of the number of men employed.

Shop Floors

The character of the shop floors has an influence on production, because it has an effect on the character of the labor that can be secured and also upon the amount of work that can be performed in a given time. The floors should be level, clean, in good repair, and not too hard. A wood-block floor seems to meet these conditions, as it can be kept clean and level, is not too hard, is long lived, and can be repaired quickly and economically. It is not cold, and, when kept clean, it is not slippery.

Straightening Metal

A certain amount of material as it comes from the mills is not sufficiently straight to be used in the condition in which it is received, if the result is to be a first-class product. Straightening by hammers is unsatisfactory and expensive, and it is prohibited by many specifications. The best results can be secured by the installation of straightening rolls for plates and angles and rolls or presses for channels and I-beams.

Marking Metal

The marking-off of the material is largely done by hand; and efficient appliances should, therefore, be provided for handling and holding the individual pieces. Jib-cranes or traveling wall-cranes are suitable for such manipulation. They should have either electric or quick air hoists, and should be operated from the floor. Overhead cranes should be provided for delivering materials to the proper zone and for removing them therefrom. Proper skids should be furnished on which the material can be marked off and stored.

Trimming and Cutting

As trimming and cutting of metal in the condition in which it comes from the mill are always required, the necessary space and tools should be provided for doing this work either before the material is marked off or before it is punched. Among the tools that might be mentioned are

angle, beam, and plate shears; coping machines for coping I-beams and channels; planers for trimming the edges of plates; planing or milling machines; and machines for ending stiffeners and chamfering them so as to fit the flange angles. Planing or chamfering machines can be designed with one head fixed and one movable, so as to permit the setting of the heads a given distance apart, whereby both ends of a number of stiffeners can be chamfered at one setting, and all the duplicate pieces chamfered while the heads are a given distance apart.

Punching

In punching, the material has to be handled in individual pieces. Jib-cranes or traveling wall-cranes should, therefore, cover the tool area; and overhead cranes should be provided for transferring the material to and from the punches in quantities. As an alternative, facilities for trucking the said material in quantities might be provided. Punches, to as great an extent as possible, should be equipped with spacing-tables of some of the various standard designs; and, where practicable, the punching should be done by spacing machines. The cost of punching a given number of holes is reduced thereby, the marking-off of the material is eliminated, and, in most cases, the cost of templet-making is diminished. It is also possible to punch a larger number of holes within a given time; and the increase in capacity of the Punch Shop would make spacing-punches economical, even if the actual cost of doing the work were the same. In addition to this, as a rule, the work is more accurate, which condition militates for economy, in that the fitting-up is facilitated.

It is probable that more small, labor-saving devices have been introduced in Punch Shops than in any other part of the fabricating plant—such, for instance, as electrically-operated gags, ball tables, punching through wooden or pasteboard templets, using a model and applying the principle of the pantagraph, etc.

Drilling

Many bridge specifications require metal of over a certain thickness to be drilled instead of punched; and often such drilling really proves to be more economical than the punching. The shop, therefore, that is going to handle heavy bridgework has to have, in addition to its punching equipment, a sufficient number of power drills to permit the drilling from the solid of a considerable tonnage of metal. If the shop is designed especially for this class of work, sufficient drills should be provided to handle from fifty to seventy-five per cent of the normal output of the shop.*

* The author predicts that it will not be many years before all important steel bridgework will be drilled solid, and that the increased cost of so doing will amount practically to zero.

Storage of Punched Metal

As it is not possible in a shop handling a large tonnage to punch or drill all of the material for a particular unit at the same time, sufficient space should be provided in the Punch Shop, or the Assembling Shop, or between them, for the storage of punched or drilled material, so that when the Assembling Shop is ready to start on the assembling of a given unit, or any part thereof, the metal can be located and delivered to the assemblers as wanted.

Assembling

The planning system should also provide for the performance of secondary operations, if necessary, before the delivery of the material to the Assemblers. Angles, plates, etc., frequently have to be bent before or after punching, and sheared edges of plates have to be planed; and the assembling work should not be held up by any of these operations.

As the assemblers handle the material in mill sizes, they require cranes for the rapid manipulation and holding of same while assembling. These cranes should, preferably, be operated from the floor. Overhead cranes should be provided for handling the finished pieces. The assembling area should be equipped with level skids at such a height above the floor as to permit of accurate and rapid assembling. It is important that the skids be truly level, so that the material assembled thereon will be straight and out of wind.

If girders are assembled in a vertical position, cast-iron bases, or something similar, should be provided for holding them during the assembling. These bases should be of such a character that the bottom flanges of the girders can be readily clamped thereto, in order to prevent the girders from falling over. The tops of the bases should be at the same elevation; and, if the girders are to be cambered, steel filling-pieces can be placed above the bases, so as to provide the camber desired.*

Reaming

Present-day specifications for bridgework provide, in the case of a large portion of the work, that the rivet holes shall be reamed after assembling. This requirement renders necessary the furnishing of a complete reaming equipment; and, in the end, economy will result if the reaming capacity is sufficient to take care of the entire output of the shop. Even though the specifications do not require it, economy will result from the reaming of all rivet holes after assembling. The reamers should be power-driven and capable of being forced up to the capacity of the drills.

Level skids should be furnished in the reaming area on which the

* The author deems the cambering of plate-girders to be absolutely unnecessary and, consequently, uneconomic.

material can be reamed, and on which it can be stored so as to remain straight and out of wind. Overhead traveling cranes should be provided for conveying the material to and from the reamers.

Riveting

Rivets should be driven as much as possible by power machines; and machines of different types should be provided, in order to be able to drive the greatest number of rivets practicable. These machines should have ample power, because tight rivets should be secured throughout the work the first time they are driven, the expense of cutting out and redriving rivets being thereby reduced to a minimum. Portable riveting machines probably give the best results, as it is more economical to move a comparatively light riveting machine than it is to shift a heavy piece of bridgework. Machines can be quickly handled and the rivets can be rapidly driven, if the machine is suspended from a wall-crane or a jib-crane. These cranes can be operated from the floor by one of the riveting gang. The bridge members can be transferred by traveling wall-cranes, overhead cranes, or trucks.

Milling and Planing

Certain members of trusses, columns, and sometimes even girders have to have their ends planed; and various machines have been built for this work, a rotary planer being generally the most economical. Milling machines, however, have been used. No matter which type of machine is employed, economy will result by having it furnished with two heads, one of which travels on the ways of the apparatus. This permits the heads to be set at a given distance apart, and both ends of the piece to be planed or milled at the same time, duplicate pieces being finished without re-setting the movable head.

Boring

Double-headed horizontal or vertical boring machines should be provided with one head traveling on the ways of the machine, so that the pin holes can be bored at an exact distance apart. For duplicate parts there will be no need of re-setting the heads.

Special Space

All bridge members do not need to go through the same number of operations in order to be completed; and some provision should, consequently, be made for keeping material that requires only one or two operations (or special operations) out of the Main Shop. To this end a special space should be provided for the fabrication of shoes, cross-frames, laterals, small struts, and other miscellaneous pieces.

Machine Shop

For the fabrication of fixed spans not many machine tools are required, the turning of pins, the planing of main members, shoes, and base plates, and the boring and drilling of holes being about the only machine operations required. If, however, movable spans are to be manufactured, a small machine shop will effect an economy of operation. Such a machine shop should be equipped with the following tools:

Boring Mill,
Grinding Machine,
Lathe,
Screw Cutter,
Planing Machine,
Slotting Machine.

Foundry

It is probable that an iron or steel foundry cannot be added to the equipment of the fabricating plant with economy, unless there is a market for general commercial work; because there are not enough castings required in bridgework to keep a foundry busy more than a very small portion of the time.

Blacksmith Shop

A Blacksmith Shop should be installed with hammers, presses, heating furnaces, etc. If the Shop is one of large capacity, there should be added a rivet-and-bolt-making plant with the necessary heating furnaces, continuous rivet-making machines, bolt machines, screw-cutting machines, and bins for the storage of rivets and bolts.

Shop Assembling

Bridge specifications in many cases provide that the members of riveted trusses shall be assembled at the shop, that the field holes shall be reamed while the trusses are so assembled, and that the members shall be match-marked before they are taken apart. This ensures the proper matching of the holes when the work is subsequently erected. The shop, therefore, should have the necessary space and equipment for assembling such trusses. Generally, it is economical rather than the reverse to do such assembling and reaming; but, if there be a large number of duplicate spans, it is uneconomical. In such a case the field connections should be reamed to iron templets, so that the members will be interchangeable. The work can be done much more quickly in this way; and, if it be carefully performed, the results will always be satisfactory. If desired, one or two of the trusses can be assembled before any of them are shipped, in order to check the accuracy of the work.

If possible, the space for such assembling of trusses should be commanded by overhead traveling cranes, and should be equipped with portable power drills, both horizontal and vertical. These portable drills will be found of great value in the shop as a supplement to the regular drilling and reaming equipment. The top and bottom chords of the trusses, after planing, should be assembled in the shop, and the holes on one side of the splice drilled from the solid whilst the chords are thus assembled, using these portable drills.

Painting

For finishing and painting, ample space and proper handling facilities should be provided. At least a portion of the space (and the larger the better) should be covered, as all of this work cannot be done out of doors during the winter. While spraying machines have been used for painting, doing the work by hand seems to give more satisfactory results.

Tracks

Ample track facilities should be provided both for the bringing in of material to the shop and for shipping it out.

Templet Shop

For economy, the templet shop should be light, well heated, and furnished with benches at one level, and with tools for planing, cutting, and boring the templet lumber and for boring and shearing pasteboard for templets. The use of the latter material instead of lumber is in the line of true economy.

Recapitulation

To summarize—the main factors are light, heat, ventilation, space, and tools; and the Planning System should result in:

Availability of material when desired.
Short and continuous travel.
Handling as few times as possible.
Keeping of men busy.
Tools of such character as to permit the securing of the proper grade of workmanship.
Competent employees to maintain such a grade of work.

CHAPTER XXXVII

ECONOMICS OF BRIDGE ENGINEERING FIELDWORK

THE economics of fieldwork in bridge building may be summed up in the following instructions to the Resident Engineer:

First. Start work ahead of the contractor so as to have all the triangulation finished and bench-marks established before he is ready to begin actual operations.

Second. Employ as small a force as practicable for attending properly to all work, especially the inspection of materials and workmanship, and retain only truly competent, energetic men. If low-grade helpers are required occasionally, obtain them from the contractor, and be sure to pay him for their time, unless the specifications or contract provide that such labor shall be furnished free of charge, as is often the case.

Third. See that the entire force is kept really busy at all times. This can be accomplished by laying out for each member thereof routine work for his spare moments.

Fourth. Before any construction work is begun, lay out on paper an economic system for carrying on the field work from start to finish, and see thereafter that it is strictly followed. Should at any time any development occur that would render advisable a modification of the working schedule, the necessary changes therein should at once be made; and all interested parties should be notified in writing accordingly.

Fifth. Never let any portion of the work drop behind, even for a single day; because field engineers are assumed to work longer hours than the staff of the designing office. Field engineers should live on the job, so as to be ready at all hours to do unexpected work and to meet emergencies, consequently they have no cause for complaint when instructed to work overtime.

Sixth. See that the home office is supplied strictly on time with the regular field reports, and send in special ones whenever such seem necessary or advisable.

Seventh. Do the testing of cement far enough in advance of its use to make sure of its having the requisite strength; and if there be any other materials to be inspected, keep the work of inspection well in advance of the demand.

Eighth. As soon as materials are received at the site, see that they are checked by the contractor at once, and report immediately to the home office any shortage discovered, in order that it may be made good before it can cause delay in the construction. See also that all material is unloaded

quickly, even if the onus of paying for demurrage be upon the contractor, because it is economic to keep the railroad officials in good humor by returning their cars quickly, and because it does not take many unloaded cars to clutter up the small, temporary yards built specially for the job.

Ninth. Give the contractor and his employees every aid that you can legitimately, short of assuming duties that are not your own—such as preparation of bills of materials, designs for plant, etc. By keeping the force in a good humor the work will be expedited, and economy for all concerned will result.

Tenth. Make all monthly estimates promptly, starting in two or three days before the end of the month, if that be necessary to accomplish the purpose.

Eleventh. For all unclassified work or so-called "extras," be sure to give the contractor in advance written instructions to do the same, and keep on file copies of all letters containing such directions.

Twelfth. Keep your progress reports and charts up to date so that you may know at a glance what proportion of each class of work on the job is finished and what proportion remains to be done.

Thirteenth. Remember at all times that the Resident Engineer is a confidential agent and not a principal, and be governed accordingly. While he has the right to discharge at any time any of his employees for just cause, he should keep the home office *au courant* with the character of the work of his assistants, in order that, if changes in the staff are to be made, the field work will not be disorganized thereby.

Fourteenth. If practicable, the field property of the Engineers, such as instruments, tapes, testing apparatus, etc., should be insured against loss by fire, theft, or the action of the elements. Sometimes this is impossible; but whether it is or not, every precaution should be taken to prevent such occurrences, not merely because of the pecuniary loss to the principals but also because of the delay to the fieldwork that would be likely to result.

Fifteenth. Copy all important surveys from the field books into a survey-record as soon as made; and never let that record be taken into the field, for its loss might cause great inconvenience and expense. When looseleaf books are used for records that might be called into a court of law, each page must be signed and dated by the writer thereof at the time he makes the record; and each such page must have a heading or title which will show beyond any doubt just what is recorded thereon and the source of the information.

Sixteenth. Log books or diaries must be written up each day, whether there was any work done or not.

Seventeenth. As a great many of the Resident Engineer's duties are of a semi-judicial character, all of his acts may at any time come under the review or inspection of a court of law; hence he must always keep this in mind when making decisions or compiling records. The latter must be full, concise, and so made as to be admissible as evidence in a suit at law.

Eighteenth. All instruments for surveying or for testing materials, when not in use, must be kept in their cases or boxes in the Resident Engineer's office. No transits or levels should be left set up around derricks or machinery where there is danger of their being struck by the said machinery or by teams, unless someone is left to guard them.

Nineteenth. The Resident Engineer must be careful not to abuse in any way the rather arbitrary power that is placed in his hands; but, on the other hand, he must not fail to act promptly in exercising the authority conferred on him, if it be necessary to do so in order to protect the interests of his client or those of his superiors.

Twentieth. See that all materials are properly stored and kept in good condition until used in the construction; and make sure that the contractor takes every precaution to prevent injury to them from fire or flood, no matter whose property they may be. If practicable, they should be insured.

Twenty-first. Make sure that no damage is done to structural steel or reinforcing bars through carelessness in handling or unloading.

Twenty-second. See that all wooden paving-blocks are ricked up in compact piles and covered so as to prevent checking; and, as all creosoted timber is very inflammable, take every precaution against its being injured by fire.

Twenty-third. Make sure that all cement is properly protected against the weather; and that no injured cement is allowed to remain on the job.

Twenty-fourth. Take great care to avoid accident in the storage or use of explosives; and see that the proper charges are used.

Twenty-fifth. Make sure that all falsework and forms—in respect to both design and quality of materials—are fit for the purpose to be served; and take the necessary steps to prevent unsightly bulges and offsets in concrete surfaces due to the yielding of forms.

Twenty-sixth. In sinking cribs or caissons do not permit the contractor to allow them to get materially out of position or tipped; but check constantly for line, elevation, and verticality until all danger of the occurrence of such errors of any magnitude is past.

Twenty-seventh. Look carefully to the building up of cribs and caissons; because, if they become badly warped or twisted, much needless expense and delay will be involved.

Twenty-eighth. Watch carefully the depositing under water of all concrete so as to make sure that it is not injured in the process of placing.

The preceding instructions, of course, could readily be extended by entering more into detail, and it is true that the Resident Engineer should study and follow closely many other directions, both written and oral, given him by his superiors; but enough has already been said in this chapter to indicate the necessity for a proper application to fieldwork of the principles of economics, what the said principles are, and how they should be utilized.

CHAPTER XXXVIII

ECONOMICS OF BRIDGE-CONTRACTOR'S GENERAL FIELDWORK

THE data for this chapter were furnished by Mr. H. K. Seltzer, C.E., of the Union Bridge and Construction Company, and, in years long gone by, one of the author's principal assistant engineers. As that Company is one of the most experienced and successful bridge-contracting firms in America, what Mr. Seltzer has to say about the economics of his specialty ought to be authoritative.

The Contractor who has been awarded a contract naturally says to himself—"I must do this work as quickly and as economically as practicable." He, therefore, gives thorough consideration to the problem of how this result can be brought about; and, while so doing, he should think not only of the profit he expects to make, but also of the fact that he must maintain his good reputation, and improve it, if possible. In order to complete the work promptly and economically, he should devote careful thought to the following subjects:

1. The Field Organization.
2. Plant.
3. Yards, Wharves, and Tracks.
4. Plans of Buildings and Plant.
5. Materials and Supplies.
6. General.

1. THE FIELD ORGANIZATION

The man in active charge should have had previous experience, either as a superintendent or engineer of construction, in charge of similar work. It is not absolutely necessary that he should have managed work equally large or important; but he should certainly be a man of character and force. He may be known as Chief Engineer, Manager, General Superintendent, or Engineer of Construction. We shall assume that he is an engineer and that his company has designated him as "Engineer of Construction." He should have the following general assistants, if the size of the work warrant it,—these men to report to him directly:

Assistant Engineer of Construction. General assistant to Engineer of Construction. In charge of field and office engineering work.

Superintendent. In charge of all foremen, including the master mechanic. The superintendent should have complete charge of work in field and should be a capable and reliable man.

Purchasing Agent. The making of all purchases should be placed in the hands of a competent man. Schedules of materials and supplies should be furnished to him by the office engineers, specifying when they will be required; and he should arrange his deliveries as nearly as possible in that order.

Accountant or Auditor. In charge of all office work outside of the engineering department. He should have charge of bookkeepers, paymasters, timekeepers, cost accountants, material checkers, storehouse men, and any other clerical help required.

2. PLANT.

The best plant obtainable should be provided in sufficient quantity to permit of the rapid execution of the work with the smallest possible number of men. Hand, air, or electrical tools should be liberally provided. There should also be furnished spare equipment, such as an extra hoisting engine, an extra derrick, and an extra dredge-bucket. On large jobs of work these extra pieces of equipment are always needed because of breakdowns. A liberal supply of repair parts for engines, pumps, pneumatic tools, etc., should always be kept on hand.

3. YARDS, WHARVES, AND TRACKS.

Sufficient ground should be obtained to provide storage room and space for the necessary operations. Existing wharves should be secured, when they can be had, and new ones built when necessary. A plan should be prepared showing the exact location of railroad tracks, storage grounds, buildings, and wharves. The arrangement, of course, will depend to a great degree on the ground and the river front available; and the cost and progress of the work will vary largely with the care and attention given to this subject.

4. PLANS OF BUILDINGS AND PLANT.

After it is known just what yard-room can be secured, and after a general ground-layout has been drafted, detailed plans of buildings, derricks, and other material-handling devices should be prepared; then the necessary bills of material should be made therefrom. These plans should include drawings of barges, pile-drivers, falsework, etc.—in fact of everything that must be built at the bridge site.

While planning buildings and barges, it is well to give consideration to the fact that, after the completion of the job, buyers may be obtained in the vicinity, provided such constructions are so designed as to meet local conditions and requirements.

5. MATERIALS AND SUPPLIES.

Schedules of materials for both construction and permanent work should be made as quickly as possible, even though the contractor may have contracted for his materials before bidding. It is important that no time be lost, in order that early shipments may be made; and, again,

prices change so rapidly that those used in bidding may not hold in every case, especially if there be considerable delay.

Materials should be ordered for shipment as nearly as possible in the order required. It frequently occurs that no special attention is given to this point; and, as a result, delay and unnecessary cost are involved.

Contracts for fuel, oil, water, electricity, and supplies should be made immediately, whenever it appears to the contractor's advantage so to do.

6. General

It pays well to have satisfied and loyal employees; and every effort should be made to provide and maintain such a force. Special care should be used in the selection of the skeleton organization, including the foreman; and these men and all other employees should be well paid, and given the best treatment possible. Where camps are necessary, they should be sanitary; and good food should be provided at actual cost.

It is well to prepare in advance a schedule of the progress of the work that it is intended to follow; and plans should be made accordingly so as to carry it out.

Daily unit costs should be kept, when possible, in addition to the usual cost system; and the foremen and all others interested should be advised how well they are found to be doing, comparisons being drawn with other work of a similar nature.

Nothing should be done by hand that can be accomplished with machinery or power tools.

In concluding this chapter, the author begs to make, for the benefit of progressive contractors, the following additional suggestion, based upon extended experience and close observation of bridge builders and their methods of operation:

It is almost always consistent with true economy to push every piece of work to completion as rapidly as practicable, even if by so doing there should be involved additional expense (of course within the bounds of reason) for outfit and labor; because the time thus saved can generally be more advantageously devoted to another job.

CHAPTER XXXIX

ECONOMICS OF CONCRETE MIXING

In the mixing of concrete there arise many economic problems, most of which are apparently of such minor importance as not to be worthy of consideration. For small jobs of work this may be true enough, but on large ones, involving great yardage of concrete, every little item of economy is well worth while.

There are two main points of view from which this question may be considered—that of the contractor and that of the engineer—the latter, of course, acting for the best interests of his client. The contractor generally is paid for concrete in place at so much per cubic yard; but there are often several unit prices arranged to cover varying conditions. Sometimes these conditions are variations in the composition of the mixture, but often they relate to the differing costs of placing and formwork. Be this as it may, though, in ordinary specifications there is nothing to induce the contractor to try to produce the strongest and best concrete practicable. This is the duty of the engineer; and he accomplishes it by stating clearly in his specifications the qualities of all the materials employed, the compositions of the different classes of concrete, how the components thereof are to be mixed, how the fresh concrete is to be placed, and how the finished work is to be protected until the mass hardens and dries—also by carefully inspecting the materials for quality, the mixing, the placing, and the protection, so as to ensure that the specifications are fully complied with.

Ordinarily there are not many legitimate major ways in which the contractor may economize in his concrete-work, as he is confined mainly to purchasing the materials as cheaply as possible, doing the mixing economically, and placing the mixture expeditiously; but occasionally he is given an opportunity to economize on the quantity of cement by the engineer's having made it proportional to the volume of voids in the aggregate, and by being permitted to use one-man stones in the mass.

There are, however, many minor ways in which the contractor may legitimately economize in the making of concrete, principally by the application of forethought in the receiving and storing of materials promptly; locating the bins in the most favorable places for expediting the work; establishing the best practicable scheme for transportation from bins to mixers and from mixers to points of deposit; substituting machinery for man-power wherever it will facilitate the work and reduce cost; designing and building forms that are not too expensive, that can be placed and removed expeditiously, and that have some salvage value; and installing

efficient and not-too-expensive apparatus for protecting the finished concrete from injury by frost or heat. Doing all these things scientifically and systematically will often enable a contractor to make money out of a job on which otherwise there would have been a loss.

Amongst the economic problems that arise in the manufacture of concrete, the principal ones are the following:

A. Best proportions of materials.
B. Reduction of voids in the aggregate.
C. Using a mixture of gravel and sand without screening.
D. Waterproofing.
E. Increasing fluidity of mixture.
F. Manner and time of mixing.
G. Amount of mixing water.
H. Use of large stones in mass.
I. Vibration and jigging of newly placed concrete.
J. Age of cement.
K. Protection of fresh concrete.

Each of these economic questions will be taken up in the above order and discussed in detail.

Best Proportions of Materials

Without knowing in advance the kinds and characteristics of the aggregates which the successful bidder will employ, it is impracticable for an engineer to specify the best possible proportions for concrete, hence he is either compelled to name three or four standard mixtures or else to specify certain maximum limits of the materials in the aggregate and an amount of cement proportionate to the volume of voids therein.

The principle to be adopted in specifying the proportions for concrete is to use a little more than enough cement to fill the voids in the sand and a little more than enough of the resulting mortar to fill the voids in the broken stone or gravel. If the mixing were perfect, there might not be good reason for these excesses of cement and mortar; but, of course, it never is. The author is satisfied with an excess of ten per cent for each case, or with an excess of ten per cent of cement above the volume of voids in the aggregate.

The most common proportions specified are those given in the following table:

TABLE 39*a*

Ingredients	Proportions				
Cement	1	1	1	1	1
Sand	2	2	2½	3	3
Broken stone or gravel	3	4	5	5	6

The mixture of 1 : 3 : 5 used to be the standard for concrete in large masses placed in the dry, but there are certain sands of such uniform grain-size that one portion of cement to three of sand will not fill all the voids therein; and, although the mortar produced will be ample in amount to fill all the voids in the stone or gravel, the mass resulting will be permeable by water, and, therefore, not first-class concrete.

The proportion of 1 : $2\frac{1}{2}$: 5 is better than that of 1 : 3 : 5, because the amount of cement is always more than enough to fill the voids in the sand, and that of the mortar is more than enough to fill the voids in the stone or gravel. It is a perfectly safe mixture but not an economic one. It is being rather widely specified today for mass concrete placed in the dry; and for small jobs it is a good one to adopt therefor.

The proportion of 1 : 2 : 4 is the common one for reinforced-concrete; and the stone or gravel should not be very coarse, because otherwise the concrete would not flow properly between the reinforcing bars, and voids might result. In view of the importance of having reinforced-concrete as perfect as possible, and of the fact that the mortar should take a firm grip on the reinforcing bars, it is not advisable to use for this purpose any concrete less rich in cement than this mixture.

The proportion of 1 : 2 : 3 with fine broken stone or gravel is the author's standard for concrete to be deposited through water by trémie or trip-bucket. It contains an excess of cement to provide for the contingency that, in spite of all precautions, there may be a slight flow of water through some portion of the concrete. The author has often had occasion to examine concrete of this mixture placed through water, and has invariably found it to be perfectly satisfactory, in fact, just as good as the less-rich concrete placed in the dry. It has been stated by good authority that this proportion makes much better concrete for reinforced work than does the standard proportion of 1 : 2 : 4; hence it might prove economic to adopt the richer mixture therefor, but it would first be necessary to educate the profession to the advisability of the innovation.

The only excuse for adopting in a specification a general clause for a 1 : 3 : 6 mixture is to save expense in a structure where the total cost has to be held down to an absolute minimum, in order to meet a limited appropriation; and then it should usually be confined to locations below ground where no frost can reach. It would be legitimate to adopt it for large anchorages where great mass is required, in which case it might have to be faced with richer concrete and employed only in localities where the climate is mild. But when special care is taken in the grading of the aggregate, a 1 : 3 : 6 proportion can safely be employed, because in graded sand the 1 to 3 proportion will fill all voids in the mortar, and the latter will fill all voids in the stone, thus producing satisfactory concrete, providing, of course, that the mixing be thoroughly done.

The 1 : 3 : 6 mixture does not always make good concrete, and the author would hesitate a long time before deciding to adopt it on any of

his constructions. He did once use it for some below-ground work on a bridge in Southern California, in which every legitimate effort had to be made to keep the total cost of structure within a limit of $200,000. There proved to be a margin of less than $500 when the bridge was finally completed and turned over to the owners.

Concretes as poor as 1 : 4 : 8 or even 1 : 6 : 10 have been used in times past in large mass-work, in order to reduce the cost of construction; but, in the author's opinion, such mixtures are not legitimate. Some tests on plain concrete beams by Wm. B. Fuller, Esq., gave on the average the moduli of rupture indicated in the following table:

TABLE 39*b*

Proportions by Weights of Cement, Sand, and Stone	Proportions by Volume of Cement, Sand, and Stone	Modulus of Rupture. (Lbs. per Sq. In.) Average
1 : 2 : 4	1 : 2 34 : 4.12	439
1 : 2 : 5	1 : 2 34 : 5.17	380
1 : 3 : 5	1 : 3.51 : 5.17	285
1 : 3 : 6	1 : 3 51 : 6 21	226
1 : 4 : 8	1 : 4.68 : 8 25	157
1 : 6 : 10	1 : 7.02 : 10.34	89

This table gives a very clear idea of the relative strengths of concrete of varying richnesses; and it is evident therefrom that the last two mixtures are too poor to warrant use in any first-class construction, also that 1 : 2½ : 5 concrete is better than 1 : 3½ : 5 concrete in the ratio of 1.33. By interpolation it may be concluded that it is probably better than 1 : 3 : 5 concrete in the ratio of about 1.2. Comparing the 1 : 2½ : 5 mixture with the 1 : 3 : 6 one, the ratio of strengths would be about 1.5.

Reduction of Voids in the Aggregate

Whenever there is a large amount of concreting to be done on a job, it will prove to be economical to study carefully the percentage of voids in the aggregate of broken stone and sand (or of gravel and sand), and to experiment in order to determine what mixture of broken stone and gravel, or of several sizes of broken stone, with the proper amount of sand in each case, will reduce the said percentage of voids to a minimum. If then this mixed aggregate be adopted, and if the amount of cement is never less than one and a tenth times the volume of voids, the resulting product will be first-class and satisfactory to all concerned, provided, of course, that only proper materials be employed and that the mixing be thorough.

As a matter of precaution, however, against carelessness or error on the part of the tester of voids, the author's specifications require that, for aggregates in which all the materials are measured separately before mixing,

there shall be used not less than 420 lbs. of cement per cubic yard of finished concrete, excluding, of course, the space occupied by any one-man stones that it may contain.

USING A MIXTURE OF GRAVEL AND SAND WITHOUT SCREENING.

If there be available and located conveniently for the work a large body of clean, mixed gravel and sand, it may prove economical to use it without screening in one of two ways, viz.:

First. By constantly making tests and finding what amount of sand or of gravel should be added to the natural mixture in order to secure the correct proportions, sifting out a supply of the material required, storing it close at hand for use, and combining it with the said natural mixture; and

Second. By using the natural mixture directly as it comes from the pit.

In either case the rule previously given for a volume of cement equal to at least one and one-tenth times that of the voids should be followed; and as a matter of precaution, the amount per cubic yard of finished concrete (excluding all embedded one-man stones) should be 460 lbs. for the first-mentioned of these two methods and 500 lbs. for the second. This is in accordance with the author's standard specifications.

Whether it is most economic to separate all the ingredients of the natural mixture and re-mix in the desired proportions, to add sand or gravel to the natural mixture, or to use the pit run, can only be determined for each case as it arises by some very careful computing, based upon the governing prices of labor and materials. In fact it might be necessary to make some actual experiments. If labor were very high and the price of cement delivered at site were reasonably low, it would probably be most economical to use 80 lbs. extra of cement per cubic yard of concrete and employ the pit run; but if cement were very expensive and labor cheap, it would be most economical to sift the natural mixture and remix.

Whether adding either sand or gravel to the natural mixture, in order to bring it to best proportions is economical, will depend upon whether the screening out and incorporating of the additional material and the occasional testing of the pit run will cost less than the 40 lbs. of cement saved per cubic yard of finished concrete.

WATERPROOFING

In certain places it is necessary that the construction should be absolutely waterproof; and whether accomplishing this is to be done by adding some foreign ingredient to the cement or by inserting a layer of burlap or other similar material covered with pitch or asphaltum applied hot is a problem in economics that has to be solved for each case by some close figuring. There are on the market several patented materials to add to concrete for waterproofing, the manufacturer of each of which claims it to be the best possible for the purpose; but, as far as the author knows, there

is nothing better than the addition of hydrated lime to the cement to the amount of about ten per cent of its volume. In any case where the condition of impermeability is paramount, it would be well to adopt both of these expedients, irrespective of cost; and the combined expedients, if successful, would effect a true economy. The question of the economics of waterproofing concrete is treated at length in Chapter XLIII.

Increasing Fluidity of Mixture

The addition to the cement of not more than ten per cent of its volume of hydrated lime not only tends to make the finished work waterproof, but also adds greatly to the fluidity of the mixture, thus facilitating the placing of it around the reinforcing bars. Certain reliable tests have shown that the addition of lime up to fifteen per cent of the volume of cement really slightly increases the tensile strength after some three weeks; and as the addition of lime does not add materially to the expense of the concrete, it is a matter of true economy to employ it. The beneficial effect of hydrated lime is partly due to the fact that it permits a reduction in the amount of mixing water without lessening the plasticity of the mixture.

Manner and Time of Mixing

Almost all concrete nowadays is mixed by machinery, and preferably in batch mixers, although some continuous mixers have been known to give good results. The strength of the concrete is augmented as the time of mixing is increased; hence it is an economic problem for the engineer, *but not for the contractor*, to determine what is the best time to adopt for mixing each batch. If the time be made too short, the attainable strength and quality of the concrete are not developed; while if it be made too long, the output per mixer is reduced and the cost per cubic yard of the finished work is increased. As the contractor is paid so much per cubic yard for concrete in place, it is evident that he always loses instead of gains by increasing the time of mixing, and that the owner, up to a certain point, is a gainer by such an increase, after which he is a loser. In respect to what that limit is, most engineers differ. Contractors would like to make it thirty seconds, but are willing to concede a full minute. The author, however, would set a minimum of two minutes. As loading and unloading the mixer require for the two operations, on an average, about 45 seconds, and as the time given to actual mixing is about the same, the total time needed per batch is one and a half minutes, but when the time of rotating the mixer is increased to two minutes, the total time per batch is two minutes and forty-five seconds, hence the output per mixer would be nearly halved. Nevertheless, the author believes that doubling the ordinary time of mixing will result in true economy for the owner. The authorities recognize that it is far better, when a truly-first-class job is required, to employ more mixers, even at a higher first-cost for equipment, and work them on a

longer schedule, than it is to try with each single mixer to produce the greatest possible yardage in the least practicable time.

Greater care and increased expense in mixing and placing are in the line of true economy; for they produce a stronger concrete and, therefore, justify the use of higher unit stresses and a consequent reduction in the concrete sections. This question is of especial importance for concrete slabs on long-span steel-bridges, and in long-span concrete-bridges.

Amount of Mixing Water

It has been the general practice to use very wet mixes, especially for reinforced-concrete. From the construction standpoint this is economical, as it reduces considerably the cost of handling and placing. Recent investigation, however, has shown that an excess of water reduces the strength of the concrete very materially, and makes the concrete porous; it also tends to cause segregation of the materials. The use of an excess of water is, therefore, false economy. The new American Railway Engineering Association Specification for Plain and Reinforced-Concrete contains the following requirement:

> The quantity of water used in mixing shall be the least amount that will produce a plastic or workable mixture which can be worked into the forms and around the reinforcement. Under no circumstances shall the consistency of the concrete be such as to permit a separation of the coarse aggregate from the mortar in handling. An excess of water will not be permitted, as it seriously affects the strength of the concrete; and any batch containing such an excess will be rejected.

Use of Large Stones in the Mass

When the contractor is permitted, under certain restrictions, to place one-man stones in the concrete in order to save mortar, he usually thinks he has a "soft snap," but sometimes this is not the case; because, in addition to their having to be carried to the site, they must be thoroughly cleaned and wetted before being placed. This placing when properly performed takes time, and is done by man power—not by machinery. Again, such one-man stones are generally boulders taken from the river bed, where they are often found covered with moss and slime, all of which has to be carefully removed. In the old days when labor was cheap and cement expensive, these one-man stones were looked upon as plums in the pudding, but today the same volume of straight, machine-made concrete will often prove less costly than the said "plums." In case, however, there is old stone masonry to dispose of, it will generally be found cheaper to use it as one-man stone in the concrete rather than to haul it away.

Vibration and Jigging of Freshly-made Concrete

During the past year there has been considerable talk about the benefits to be derived by fresh concrete through vibration and jigging; but the

truth is that, while such treatment sometimes augments the strength, it often has the opposite effect. From the résumé of an able paper by Prof. Duff A. Abrams entitled "Effect of Vibration, Jigging, and Pressure on Fresh Concrete" the author extracts the following:

The indications of the vibration and jigging tests should not be misinterpreted. The tests show that, *after the concrete is properly placed*, these methods of treatment do no good, and may be harmful, if too severe or too long continued. However, there can be no doubt of the value of such methods *for getting concrete into place* in intricate forms and around reinforcing bars. The tests are of value in showing that this is the only desirable function of such treatments.

The tests show that, with jigging, high strength may be secured with drier mixes than would otherwise be feasible. It is a matter of common experience that concrete of drier consistency (and consequently higher strength) can be placed by means of jigging or vibration than would be possible by the usual methods.

It is clear from these tests that if tamping, vibration, or pressure on fresh concrete is to be effective in increasing its strength three factors must be kept in mind.

(1) We must take advantage of the fact that with these methods the concrete can be placed and finished drier than with ordinary methods.

(2) Excess water which is brought to the surface must be removed.

(3) We must take advantage of the fact that aggregate of a coarser grading may be used when such methods are employed than would be practicable otherwise.

The advantages to be gained under (3) are due to the fact that, up to a certain point, a plastic mix can be secured with a smaller quantity of water, if the aggregate is as coarse as practicable. Unless these precautions are taken, tamping and vibration are of doubtful value.

Age of Cement

Prof. Abrams has lately proved that the ultimate strength of concrete in compression is a function of the age of the cement at time of mixing, the fresher it is the greater the strength. This is at variance with the idea which governed previously, viz., that cement is better for a little aging; but we are learning these days that many of our old ideas about cement and concrete were incorrect.

In Bulletin No. 6 of the Structural Materials Research Laboratory of Lewis Institute, Prof. Abrams presents a number of diagrams showing the percentages of loss of strength in compression for concrete by using cement that had been stored from two to twenty-four months, as compared with that manufactured with perfectly fresh cement; and the amounts thereof are surprising. They average fifteen per cent for two months and fifty-five per cent for twenty-four months, with almost proportionate percentages for intermediate periods. However, the loss is not quite as bad as these figures would show, because the experiments prove that the concrete tends to recover its strength with age. For instance, when two-months-old cement is employed, the loss is nineteen per cent at one month and only eleven per cent at two years; and when two-year-old cement is used the loss is fifty-six per cent at one month and only forty-two per cent at one year.

The large amounts of these losses indicate clearly the desirability (from the standpoint of the strength of the finished product) of using cement as soon after grinding as is practicable. It must not be forgotten, however, that cement which has had no aging is likely to be deficient in regard to soundness; so that its testing in respect to this requirement should be extremely thorough. The author for many years has insisted upon twenty-eight day tests on cement whenever that arrangement would not delay the progress of the work; but on the strength of Prof. Abrams' findings, he would now be willing to accept on seven-day tests any standard cement with which he is acquainted.

Again, the author has sometimes made arrangements for testing and storing large quantities of cement at the manufactory, so as to have it available when needed; but he may now be obliged to forego this practice.

Prof. Abrams' experiments were made on sacked cement; and he intends duplicating them upon cement shipped in bulk. Possibly the result of the new tests will show that keeping the cement in large masses will postpone somewhat the sudden drop in strength of concrete which occurs with cement two months old. It would be interesting to learn what the loss is on cement one month old; and it is to be hoped that Prof. Abrams will add to the value of his very useful experiments by settling this point.

PROTECTION OF FRESH CONCRETE

The protection of fresh concrete is an economic matter, as far as the contractor is concerned, because, according to all properly drawn specifications, he will have to make good at his own expense all damage done to the concrete through excessive cold or heat, or any other cause, before the final acceptance of the finished construction. It is also a matter of economy to the owner, because if the concrete is at all damaged, all the repairing that the contractor may do, short of entire removal and rebuilding, will fail to make the job as good and satisfactory as it would have been, had no such misfortune occurred.

GENERAL REMARKS

From the point of view of both the engineer and his client, it is truly economic invariably to obtain concrete that is first-class in every particular—strong, solid, smooth, and hard—no matter how much it may cost; because failure of any kind in a reinforced-concrete structure is likely to be both serious and expensive. Many large bridges of that type, which were constructed under contracts secured through competitive bidding, show after a few years signs of disintegration. The ultimate result of such work is that the structure sooner or later will require either extensive repairs or entire rebuilding. The causes of such failures may be attributed to objectionable methods used by contractors in order to cheapen the cost of han-

dling and depositing the concrete—such as chuting it long distances, using gravity mixers, allowing insufficient time for mixing, putting too much water in the mixture so as to avoid most of the expense of tamping and working the aggregate back from the face of the forms, permitting the employment of too large pieces of broken stone which fail to pass between the reinforcing bars, laying concrete in freezing weather, and placing concrete under water when there are available other methods which would permit depositing it in the air at a little greater cost.

At the time when the "Final Report of the Joint Committee on Concrete and Reinforced-Concrete" was presented, the above condition prevailed, but since then much important research work on concrete manufacture has been done, and there are now several publications which give valuable data and point the way to better concrete structures—notably the records of the elaborate series of experiments made by Prof. Duff A. Abrams. But as this treatise is intended to cover solely the field of economics and not that of general engineering practice, the reader is referred for further data on the manufacture of proper concrete to the well-known "Report" before mentioned; the "Transactions" of the American Railway Engineering Association, the American Society for Testing Materials, and the American Society of Civil Engineers; and the "Bulletin" of the Structural-Materials Research-Laboratory of the Lewis Institute.

The author is indebted for a number of valuable suggestions on concrete manufacture to his friend, J. J. Yates, Mem. Am. Soc. C. E., Bridge Engineer of the Central Railroad Company of New Jersey, and Chairman of the Committee on Masonry of the American Railway Engineering Association.

CHAPTER XL

ECONOMICS OF ERECTION

THE data for this chapter were furnished by the author's old friend, Frank W. Skinner, Mem. Am. Soc. C.E., who, for a number of years, made a special study of bridge erection in all its details, and gave the results of his findings in numerous lectures to engineering students of the leading universities and technical schools of this country. Much of the valuable material that he collected was published in *Engineering Record*, of which paper for a long time he was the chief editor and leading spirit. The author feels that he is exceedingly fortunate in obtaining for the writing of this chapter the assistance of an eminent engineer who has made such a thorough study of bridge erection in all its ramifications. There are very few engineers who are fitted by both experience and temperament to discourse scientifically and practically concerning the essentially specialized subject of the economics of erection; and of these there is probably not one who possesses such a grasp thereof as does Mr. Skinner. What follows is given as closely as practicable in his own diction.

The erection of an important bridge is a function first of the design, second of the location, and third of the available equipment; and its economics are directly related to these fundamentals, variations in which materially influence the total cost of erection of spans of the same length, the same carrying capacity, and the same type. The recognition of these facts has been such that, in this country, the art of bridge erection, particularly of steel spans, has been highly specialized—to an amount comparable with that of the fabrication of bridge superstructures in great shops fitted with costly tools and used almost exclusively for this single purpose. It has resulted in the development of a clearly-defined, standard practice and the perfection of a number of methods for securing essentially the same effects under different conditions and with different appliances arranged to secure the maximum safety and rapidity with the minimum total cost.

These methods, and the special equipments that have been devised for them, make it possible for a given structure to be handled in several radically-different ways, which may either show in some cases comparatively-uniform results, or, generally, will indicate a decided advantage for one method over all the rest.

The physical and mechanical combinations can be readily analyzed so as to give a limited number of principal cases that will here be classified

for the general and comparative economics, so that, by intelligent development and modification, they will cover the field within the limits of justifiable construction.

The subjects considered are divided on broad lines into steel and concrete; long, medium, and short spans; and high and low structures.

Steel Bridges

As far as possible all items of fabrication should be completed at the bridge shop, so as to reduce the amount of assembling, fitting, and riveting in the field to a minimum. No work should be done at the site which can be performed at the shop; and no work should be done on the structure itself that can be performed ashore upon the separate or combined pieces before erection. Standard plant and equipment should be used; the largest possible proportion of work should be done by machinery and power; and the most skilful and experienced labor available should be employed in conformity with the equation of the different costs to a minimum for the completed work, including salvage, rental of plant, cost of transportation, installation and removal of plant, and the greater or less importance of extra speed (as in case of danger from floods), always considering the fundamental requirements for absolute safety and the excellence of the finished work.

It is assumed that the methods, plant, equipment, and service best adapted to the type of structure and the given conditions are available, and erection with them will be considered the economic method; but this decision, of course, is subject to modification when the problem is complicated by artificial conditions or by sudden emergencies that make changes of details, methods, or equipment safer or more practicable, as, for instance, when labor troubles, difficult transportation, scarcity of materials, or accidental physical developments make the original preparation susceptible to delays and to important changes of conditions that may have a vital influence on erection operations; and radical changes in the original programme are sometimes necessary, in order to prevent large increases over the proper estimated cost.

Girder Spans

These are plate girders or riveted trusses of such dimensions that they can be erected complete as units, usually not exceeding 50 or 60 tons in weight, 100 feet in length, and 15 feet in depth. Even these limits are likely to be excessive for transportation by rail from the fabricating shop, the size generally being limited by bridge and tunnel clearances and track curvature, and the weight by twice the capacity of each of the available cars.

If the girders can be shipped entirely by water, the limits for transportation are very greatly increased; and under special circumstances they

may be equally extended for erection. Ordinarily the latter is a very simple process, effected by one or by two derrick cars or wrecking cars that unload, transport, and erect the girders in position in single or successive operations at a direct cost of as little in pre-war times as $1.00 per ton.

Derrick cars or their equivalent are not always available in remote localities, and are not generally possessed, except by railroad companies, bridge companies, and important contractors. In their absence the girders can be handled, usually less advantageously, by derricks, gin-poles, and various combinations of jacking, blocking, rolling, and other operations that may be devised, modified, and combined according to the experience and ingenuity of the erector; and although theoretically they are more expensive and less efficient, they may often be more advantageous than the provision of high-class equipment under disadvantageous conditions.

VIADUCT ERECTION

Viaducts generally consist of two or more lines of plate or lattice girders from 30 to 100 feet long on towers up to 100 feet high, although these dimensions have been considerably exceeded in infrequent cases. The ideal method of erection is by means of a derrick traveler, sometimes called a mule, installed at grade at one end of the viaduct and having a reach long enough to erect one tower and one connecting span in advance; after which it moves forward on the completed portion of the structure, erecting panel after panel as it proceeds. Such travelers have at least two main booms of great length and one or more auxiliary booms for hoisting, swinging, and placing the steel that may be delivered either at grade or on the surface of the ground. With a large number of duplicate heavy spans, such as occur in elevated railroad construction, this class of metalwork can be erected with great rapidity, a record of 1,000 lineal feet of double-track structure per day having been made in Brooklyn with one traveler and crew, independent of the preliminary distribution of steel and the subsequent field riveting.

Where the girders have been too long or too heavy, or the towers too far apart for erection with a boom of practicable length, cantilever travelers have been successfully employed. They have been of different types, usually having an elevated horizontal boom overhanging the wheel base by the length of one tower span and one connecting span, and equipped with trolley hoists for unloading material from cars on the tracks in the rear and transferring it to the required position in advance for assembling in the structure.

In a few cases where there has been a great length of viaduct of substantially uniform height above the surface of the level ground, the structure has been erected by a strident gantry traveler, moving on a surface track and provided with several sets of hoisting tackles to handle from the ground (where they were distributed) the span and tower members.

Medium Spans

Medium spans up to 300 or 400 feet long, and including some short spans, when for any reason it is impracticable to handle the trusses as complete units, are generally, in the case of new work, best erected on ordinary framed wooden falsework by means of one or two derrick cars.

When there are pile foundations which can be placed in advance, the derrick car can rapidly put in the framed-falsework bents; after which two derrick cars, one at each end of the span, if two are available, can place the floor system, assemble the lower chords, erect the web members on them, and finish the erection by placing the top chords, top laterals, and sway bracing as the cars retreat from the center to the ends of the span.

For heavy structures where the top chords involve too great a load for a pair of derrick cars, or where locomotive cranes are used instead, they can be supplemented advantageously by a simple gantry traveler to handle the heaviest members.

It is entirely practicable to execute the erection wholly with the gantry traveler; but that traveler is costly to construct, difficult to transport from job to job, and not as rapid or economical as derrick cars when the latter are available.

Alternative Methods

If the elevation of the span is exceedingly high above the water; if there is great danger from ice or floods; if the bottom is very treacherous or difficult; if the current is too fierce; or if the space underneath the span must not be obstructed by falsework (as when it is required to be left open for navigation or for heavy city or railroad traffic beneath), falsework becomes too dangerous or expensive or is wholly inadmissible, and some other system of erection must be devised.

The most common method is by cantilever erection from each end of the span, the truss members being made heavier or temporarily reinforced until the center-panel connections are made and the structure is transformed into a simple, self-supporting span. This method involves either the provision of special anchorages and counterweights or the erection of alternate spans in advance so that they may serve as anchorages. Cantilever erection is always objectionable when it can be avoided, because it is much slower and more costly than falsework erection, and as there is greater danger of injury to the uncompleted structure by sudden storms or from various accidents than there is when falsework is used.

Sometimes falsework of various types can be provided eccentric from the alignment of the bridge, and the permanent span may be erected on it by the ordinary method—then, when complete, moved transversely to the required position and permanently seated on the substructure.

The method of protrusion is occasionally employed abroad and has infrequently been adopted in America. When it is used, the span is erected

complete at one end of the bridge and as nearly as possible at the required level, the rear end is counterweighted, and (especially if there is only a single span) the forward end is extended by a temporary pilot truss, when such extension is necessary. Each span is pushed forward longitudinally on rollers until the forward end or extension takes roller bearing on the next pier, so as to support it during the farther advance until the span comes to position and is lowered to bearing. If several spans are erected in the same way, they are temporarily connected to form a continuous-girder structure during protrusion.

Where a number of spans of substantially uniform and moderate lengths are constructed in one bridge, they have, under certain circumstances, been erected on a platform suspended from a temporary, overhead, movable span that travels from pier to pier as the work progresses.

Where the ground under the bridge is accessible, unobstructed, and comparatively level, and when materials can be delivered there, it may be possible, as has occurred in some cases, to distribute the steel in advance at low level, erect the spans there complete in the proper alignment, and raise them to required position and elevation as the piers are built up.

Considerable use has been made of the floating method whereby the spans, having been erected in the usual manner on shore or on falsework built in comparatively shallow and sheltered waters, have been transferred to the decks of scows, towed to position between the piers, and aligned with their seats on the latter, usually at low-level elevation, and lowered to place by the use of water ballast or tidal fluctuations or both.

In very difficult and unusual conditions, as in some mountain railroads, temporary suspension spans have been built, the permanent spans erected on them, and the suspension spans removed.

The above methods and their combinations, variations, and modifications have all been successfully used on different occasions; and together they cover the principal features of ordinary erection for medium, long, and short spans. Any one of them under special conditions may become the economic method for erection in that case; but where it is practicable, the method of erection on ordinary falsework with a derrick car or traveler is likely to be most desirable and economic.

Long Spans

Up to about 700 feet in length and 150 feet in clear height, spans have been erected on framed-timber falsework, which, of course, for such extreme dimensions becomes very costly, but permits more rapid and satisfactory assembling of the span than do the other methods, thus making presumption of economic desirability for this method when it can be used.

The very fact that conditions require spans of more than 400 feet is quite likely to indicate deep water, bad bottom, swift current, great height, or wide, unobstructed openings between the piers that will make falsework impracticable.

In such cases, as in medium spans, the most common solution is erection by the cantilever method, generally counterbalancing the cantilever arms from previously-completed portions of the permanent structure, building both cantilevers of the same spans simultaneously, and connecting their extremities by an independent suspended span designed to resist cantilever stresses developed by erection and to function subsequently as a simple span.

Cantilever spans have been erected up to a length of 1,800 feet with results which indicate that the same method can be extended to lengths several hundred feet greater. Serious consideration has been given to the practicability of constructing cantilever spans up to 3,000 feet in length, which length approaches the present limit set by the strength of materials; and such a detailed design would, of course, involve the development of complete cantilever-erection plant. In this case it is probable that the assisted cantilever method or some modification of it, such as has already been adopted in some large cantilever spans, would be employed, whereby temporary supports would be placed under the cantilevers between the permanent piers, thus greatly reducing the erection stresses.

The length and weight of the members of the suspended trusses in a cantilever span necessitate their erection by a heavy traveler, and, if accomplished by the cantilever method, produce a great increase of the stresses developed by the erection of the cantilever arms proper. In order to avoid these stresses, and for other reasons, the suspended span has in some instances been erected complete at a low level, towed to position under the ends of the finished cantilever arms, hoisted up, and connected.

In another case of a long-span cantilever, the ends of the cantilever arms were connected by a light, temporary suspension bridge on which the permanent suspended span was erected.

The difference in local conditions, type, and other features may make any one of these methods the most economic one for the given case, so that for an extremely large structure, all of them will probably have to be analyzed for the final comparative determination.

Suspension Bridges

Short-span and temporary suspension bridges may be designed with main cables, each composed of one or more twisted wire ropes extending continuously from anchorage to anchorage, that can be delivered complete at the site, and, with ordinary appliances, pulled across the river from tower to tower and from tower to anchorage, secured in place, and adjusted, after which the erection from them of the remainder of the superstructure is comparatively simple and easy. Generally this is the most economic method.

For all long-span suspension bridges, the universal method of erection has been the preliminary construction of falsework suspension bridges,

erected as described for short-span and medium suspension bridges, and their use as suspended falsework for the construction of the main cables built up from straight wires spliced to form continuous lines reaching around and around the anchorage pins and adjusted to the proper catenary curve and elevation as they are successively laid. These are grouped first into strands and then into the main cables; and the stiffening trusses and the roadways are erected on them by means of simple travelers.

Suspension bridges having eye-bar chains could be erected in a similar manner from a temporary, suspension-falsework platform.

Arch Spans

Very short arch-spans should be erected like girder spans, and handled as units by any convenient apparatus. Medium and long spans are generally erected on falsework or by the cantilever method. As in the case of ordinary truss spans, erection on falsework is economic when practicable; and for arches of the plate-girder or solid-rib type this method can hardly be replaced by any other, when the spans are of any considerable length. Care must be taken, however, to brace the falsework thoroughly so as to resist the oblique stresses and thrusts that are produced by unequal loading as the inclined sections of girders are assembled, unless particular pains are taken to arrange them so as to maintain balanced reactions throughout the erection. Plate-girder arch-spans up to 510 feet long have been erected on falsework.

Truss arch-spans may be erected by the cantilever method, which has been employed successfully for most of the large spans and up to the greatest present maximum of about 1,000 feet clear opening. By this method the top chords of the semi-spans which are built simultaneously are tied back to sufficient anchorages with adjustable connections so that the spans may be revolved around their skewback hinges in order to make the center connection at the crown.

Erection Plant

A very important feature in the economics of bridge erection is the design and operation of the special plant provided for handling the heavy members in the field. It has been found good, economic practice to expend large sums in the construction of plant for the erection of a single structure, and for the equipment of the field force with special machinery and power appliances of great capacity. This apparatus is so costly that none but the most important construction-companies keep it in stock; and the existence of available plant of this nature is often an important factor in the design of the structure and in the award of the construction contract. Among the important standard appliances for bridge erection are derrick cars, derricks, hoisting engines, and riveting machinery that, in general, are applicable for

all very large jobs, and may, therefore, constitute a part of the regular equipment, the same as do the tools in the fabricating shop.

The cost of their transportation, installation, and removal is, however, a special feature that must be considered in the determination of the economics of each structure. For very-long-span trusses there are necessary enormous steel travelers that, with their equipment, may sometimes weigh 1,000 tons; and, when the job is finished, they are likely to be unsuitable for future work and of comparatively small value for salvage.

For very heavy work it is also necessary in some cases to provide structural-steel falsework and to employ considerable ingenuity in its construction from portions of the permanent structure afterwards to be erected. The repeated use of special erection-metal and its availability for other purposes after the finishing of the job are important elements in the economics of the problem.

As the principal members of long spans have attained a maximum weight of more than 100 tons each, it has been necessary to provide special methods of handling them and of securing them to the hoisting apparatus; and considerable sums have been spent in the construction of steel yokes, clamps, beams, and other special devices intended solely to provide rapid and effective connections to these pieces and to enable them to be accurately and safely handled. Such appliances greatly reduce the amount of hand labor and justify considerable preliminary expenditure.

Replacing Steel Bridges

The replacement of steel bridges almost always involves the maintenance of traffic on the bridge and often of navigation below the structure during the process of reconstruction. In most cases the new structure is on the same alignment and nearly or quite at the same elevation as the old one; and frequently the old substructure is satisfactory with minor modifications to receive the new superstructure. It is often difficult, and sometimes very expensive, to divert the traffic from the old structure while the new one is being erected; hence the problem of reconstruction, especially of long and high spans, thus becomes one of the most difficult and expensive that are likely to be encountered, and the economics vary so greatly that no general determination can be made, necessitating that they be investigated independently for each structure.

For short spans where the whole span or its single complete girders or trusses can be handled as units by travelers, derrick cars, or other apparatus traveling on the ground alongside, or on the structure itself, and especially where the bridge is a double-track railroad structure, it is usually comparatively easy to divert traffic to one track, and to remove the old structure piecemeal, putting in the new parts as fast as the old ones are taken out and gradually rebuilding the entire superstructure.

For viaducts this may frequently be done with two derrick cars handling

the girders together, one at each end, provided the towers do not need removal; but in the latter case, particularly if the viaduct is very high and the connecting spans are long, the problem becomes one of great difficulty and expense.

This replacement can be, and has been, accomplished by supporting the structure on falsework; by suspending it from overhead spans long enough to reach from tower to tower, and carrying successively both old and new structures; and by providing new towers, or portions of new towers, before the old towers are wholly or partly removed.

All of these methods are likely to be slow, hazardous, and expensive; and they must be very carefully planned in detail for each structure considered.

When the old and the new structures can successively support each other during the construction, or when it is possible to by-pass the traffic, or to transpose old and new structures transversely in sections, or when it is possible to build new towers intermediate between the old ones or adjacent to them, the difficulties are likely to be considerably diminished, but such favorable conditions do not frequently prevail.

As the economics of design necessitate a rapid increase of span-lengths and weight with increasing height of track, the units handled in reconstructing a high viaduct become very large, and the derrick-car method and the translation of the structure as a whole are likely to be impracticable.

Replacing Short Spans on Old Substructure

Several methods of replacement of short and medium-length spans have been devised and repeatedly executed until they are to a certain degree standardized; and under ordinary conditions they may frequently be selected by inspection and modified, if necessary, to suit the special conditions and requirements of the case. Under such circumstances the method that provides the most simple and rapid operations with the least temporary construction is likely to be the most economic.

Where the spans are over water that is not too swift nor too much obstructed, and where navigation permits, it is frequently possible to place both the old and the new span on barges or their equivalents, and, at the given time, disconnect the old span from its original support and carry it immediately transversely out of position, and simultaneously or successively move the new span into the former position of the old one, seating it on its permanent foundations, the old span being removed to any suitable location. This system is frequently used for drawbridges; and under favorable circumstances the entire operation can be concluded with only a very short interruption to traffic on the structure. The spans are usually raised and lowered by regulating the amount of water ballast in the barges.

Replacing by transverse displacement has been successfully accomplished for spans up to 200 feet or more in length. By this method the

new span is completely erected as close alongside the old span as possible, and the floor is placed on it ready for traffic. Upper and lower sets of track-rails, transverse to the bridge axis and extending completely under and beyond both the old and the new span, are placed beneath the ends of both spans, and are separated by live rollers. Both the old and the new span are seated on the upper track-rails and are moved, usually simultaneously, transversely until the old span is carried clear of the alignment, the new span following it up until it occupies the required permanent position, when it is lowered to its masonry seat and the old span is dismembered and removed. The operations are accomplished with hydraulic jacks and power tackles; and the work can sometimes be done so rapidly as to involve only a few minutes' interruption to traffic. The rapidity of the work is often an important economic consideration that may outweigh considerable extra expense in the way of labor, material, and equipment.

For spans of more than 100 feet, it is often desirable to support the old span on falsework that is designed also to carry the new span during erection, and, after the removal of the old trusses, to erect the new span on the falsework, frequently while the traffic is carried on the old floor that is still supported by the said falsework.

If the old span is strong enough to sustain the new span during construction, plus the weight of the minimum traffic necessary, the new span may be so designed as to permit its construction by this method; and, after completion, it may, if necessary, be moved a short distance either horizontally or vertically, in order to bring it into the exact alignment after the old structure has been removed.

In other cases the new span may be designed with transverse dimensions, greater than those of the old structure, that enable it to be built outside of the latter and practically independent of it, so that the new construction may be substantially completed while traffic is maintained on the old structure; after which it will receive traffic and will support it and the weight of the old structure whilst the latter is removed piece-meal.

Replacing Long Spans

Comparatively few very-long spans have been replaced by new ones in the same alignment. The principles governing the operation are substantially those developed for moderate-length spans, with the exception, perhaps, of the application of the cantilever method, by which the trusses of new long spans have been built out as cantilever arms which clear the old structure and are self-supporting during erection.

If traffic can be diverted from the old structure during its replacement, the old trusses will be likely to be found strong enough to support the new span during erection; and, if there are more than two lines of trusses, the old ones may be successively replaced by the new ones.

In the case of a suspension bridge, the factor of safety of the cables is

such that, if well designed, they are likely to outlast the roadway and trusses, consequently the latter can easily be replaced without other support than that afforded by the cables themselves.

For spans approaching the present limits of lengths, the necessary practice is for such efficient construction, capacity, and maintenance that there is little prospect of any need for replacement of the superstructures; and, should it become necessary, there would probably be little choice of methods. Any scheme that would be possible, however, should be entitled to favorable consideration.

Erection of Concrete-Girder Bridges

Concrete-girder bridges are usually concreted *in situ* by ordinary methods and with standard equipment involving regular operations that do not present any special economic features. There may, however, be a decided economic consideration in the question of using pre-cast members for such structures. The large capacity of derrick cars and of other equipment that is available makes it possible in some cases to cast long girders and heavy floor slabs in multiple, in the Contractor's yard, remote from the bridge site and under conditions more favorable to good work and economical operation than those existing at the latter, and afterwards to transport the well-seasoned units to position and set them in place ready for service. This method is especially applicable to railway structures.

The economic considerations are likely to involve not only freight cost but also quality of structure and rapidity of operations as affecting the interruption of traffic.

Concrete Arch Spans

Arch spans must be concreted *in situ* on forms rigidly supported against settlement and provided with devices for striking the center so as to swing the arch free of its construction support when the concrete has sufficiently hardened. For short spans, low heights, and where obstructions underneath are permissible, the forms can usually be most economically supported on ordinary falsework-bents.

Where the height is great, or where passage must be maintained underneath for stream flow, navigation, or traffic, it is often necessary to carry these forms on arch-center trusses. For short spans these may be of either timber or steel; but for medium and long spans they are almost always constructed of riveted steelwork, usually arranged in sets braced together and often moved transversely from side to side of a wide bridge and longitudinally from span to span as the work progresses, thus making one set suffice for many places.

CHAPTER XLI

Economics of Maintenance and Repairs

Although in years long gone by the author did considerable work in the line of examination and repairs of old bridges, he feels that he can no longer consider himself an expert therein. It is a case of *tempora mutantur;* for the methods employed today in bridge repairing are essentially different from those that were in vogue some two or three decades ago. For this reason the author appealed to several old friends in his specialty, who are authorities in this class of work, to furnish him data for the writing of this chapter. Several of them generously complied with his request, viz., Mr. Chas. F Loweth, Chief Engineer and formerly Bridge Engineer of the Chicago, Milwaukee, and St. Paul Railway System, Mr. Carl S. Heritage, Bridge Engineer of the Kansas City Southern Railway Company, of which line the author is, and for some two decades has been, the Consulting Engineer, and Messrs. J. G. Chalfant and V. R. Covell, respectively County Engineer and Deputy County Engineer of the County of Allegheny, Penna. For their truly valuable aid the author desires to extend to these gentlemen his hearty thanks and his deep appreciation of their kindness and courtesy.

A rule of practice which the author established for his own guidance fully a quarter of a century ago seems to have found favor with the profession, viz., that any old bridge, which, in either main members or details, would be overstressed by the actual live loads passing across it, or likely soon to traverse it, not more than fifty per cent in excess of the standard intensities of working stresses employed in designing new structures, may safely be allowed to remain in use. If overstressed much more than this, it should be removed and employed at some other location where the traffic is light, or else scrapped. Exception was made in the case of plate-girder spans; for these could be relied upon to give ample warning of failing strength by rivets working loose. A plate-girder span when greatly overloaded will not collapse suddenly as will a pin-connected or even an open-webbed-riveted span.

A favorite economic expedient of the author's used to be to convert two old duplicate bridges into one and put in a new one at the crossing left vacant. This scheme was specially applicable on long lines of railway where standard I-beam and deck-plate-girder spans were used.

The more crudely a bridge was designed the more difficult it is to rein-

force it so as to make it carry satisfactorily heavier loads than those for which it was proportioned. In fact, the character of detailing employed previous to the nineties, in which decade the science of bridge design began to be established, was so outrageously unscientific as to lead the author to suggest the axiom that "the best way to repair an old bridge is to throw it into the scrap heap and build a new one."

Quite often bridges are repaired, which, from the standpoint of true economy, should be relegated to the discard. One such case of some importance occurred in the author's practice in the late eighties. It was an old Post Truss bridge across the Missouri River at Fort Leavenworth, Kansas, which had been seriously injured by the burning of a large portion of the wooden floor. The total cost of the repairs was somewhat in excess of one hundred thousand dollars—an amount greater than the sum total of all the subsequent incomes from both railway and highway traffic. In extenuation of his action in repairing this structure, the author might mention the fact that he was not consulted about the economics of the case or the advisability of repairing, but was simply given the job of engineering the reconstruction of the damaged structure. However, he is not sure that, at that stage of his career and in those days of primitive bridge design and construction, his judgment was far enough developed to enable him to come to a truly economic decision, had the problem of the economics of the case been submitted to him.

The method of solving such a problem is to estimate upon a liberal basis the probable cost of the repairs, and upon a conservative basis the probable duration of life of the repaired structure, also the probable costs of an entirely new bridge both at the date of consideration and at the expiration of the said life. If the latter cost, plus the cost of the repairs with compound interest thereon up to the time of the renewal, plus the *net* cost of removal of old structure (i.e., cost of the work less scrap value) is smaller than the *net* cost of immediate removal, plus the cost of a new structure, built immediately, plus compound interest on the sum of these two costs at the assumed later date, plus the small value of the deterioration of the new structure in the interval between the said two dates, then the repairs will be warranted.

It is evident that the correct determination of the answer to any such economic question demands wide experience, sound judgment, correct vision, and a practical acquaintance with the theory of economics.

In the old days a large portion of the work of bridge examination and repairing fell to the lot of the consulting engineers; but such now is far from being the case, because it is only for very large and important structures, or those having movable spans of a complicated character, that the independent specialists are retained on repairs and reconstruction. Such work is ordinarily done by the bridge engineers regularly employed by the railroads, the states, and the municipalities; and these men have become exceedingly expert therein.

So much for the author's views upon the subject under consideration; and now for those of his before-mentioned friends.

In respect to Mr. Loweth's contribution to this symposium, on Feb. 6, 1920, he wrote as follows:

MY DEAR DR. WADDELL,

Replying to your inquiry regarding "Bridge Maintenance and Repairs," I would refer you to a paper which was somewhat hurriedly written in the fall of 1918 for the annual convention of the American Railway Bridge and Building Association, entitled "Carrying Bridges Over," and which covers in general the matters referred to in your letter. This paper was written during the stress of war times when steelwork was very difficult to get and the need of economies was urgent; hence it was necessarily quite hastily prepared, and I feel that for usual conditions the views expressed therein should in some respects be modified.

My experience on this road * has been generally one in which it was comparatively easy to take care of the replacement of the lighter bridges. We have a great many branch lines on which the traffic is necessarily light and where in many cases it will always remain so. There was little loss, therefore, in taking a light bridge out from a first-class line and placing it in a second-class or third-class line where it would serve just as useful a purpose, at least for many years, as a bridge of the heaviest classification.

To do this involved charges to Capital Account on the lighter lines. During war times we were not in a position to assume the charges to Capital for improvement of the said lighter lines; and the difficulties of getting new steelwork, to say nothing of the very-rapidly-increasing cost, resulted in a new condition, hence we looked into the matter of strengthening bridges in place more fully than we had done previously, with the result that we did more of that work than we had even thought of doing before that time. All of this work was not uniformly satisfactory, as you can readily see, because some structures did not easily lend themselves to strengthening in a manner at all suitable from the designer's standpoint; but by the exercise of judgment and some courage, and at the same time by ignoring some of the refinements of calculation, we arrived at results that produced economies, or what may be equally important, the deferring of major expense even at the loss of ultimate economy.

The curse of the poor is their poverty, and the railroads have sometimes been, and to a large extent are now, in the position where present economy is perhaps more to be considered than ultimate economy.

Just as an illustration of what we are up against, we have on one of our second or third-class lines four rather large bridges which have been carried to their limit. The substructures are very old and small, and the superstructures also are quite old. It is an exceedingly difficult matter to strengthen them—in fact, we can only hope to remove the most glaring defects; and anything we can do in that line will permit of using only slightly heavier power than the quite-light power now being operated over that division.

Our program for strengthening would involve a cost of about $74,000. To replace the bridges with new structures would require probably more than $600,000. It seems too bad to spend so large a sum on these old structures; but assuming that we could carry them along safely for only four years, that would make an annual cost of only $18,500, exclusive of interest, and we should have saved an expenditure of over $500,000, the interest on which would amount to at least $30 000 a year. In this instance a new structure could be credited with a considerable amount for the greater safety and other considerations incident to the better bridge, but I think we shall have to decide favorably on the extraordinary repairs for what will probably be but a short period of usefulness.

* Chicago, Milwaukee, & St. Paul Railway System.

We are now aiming to restrict fiber stresses in all structures to the lower limit indicated in the paper, with the understanding that some features of the design or other conditions may make even this lower limit inadmissible and *vice-versa.*

The more important parts of the paper mentioned are as follows:

General Considerations

In the maintenance of bridges there are two general considerations to be observed:

1. Safety in carrying the necessary traffic.
2. Economy—i.e., obtaining the maximum life from the structure at reasonable maintenance cost.

On all railroads which are twenty-five or more years old, there are usually a number of light-capacity bridges which impose more or less restrictions on the train loadings that can be handled over the lines. This is a very serious problem on railroads which have many bridges that were built during the eighties and early nineties.

New bridges are generally designed for the heaviest engine and car loadings in existence at the time. In proportioning them there is, however, a certain margin between the unit stresses which are used and the maximum unit stresses which the material can safely carry. This margin provides an allowance for some future increased engine and train loadings, in addition to the contingencies which are usually embraced by the term "factor of safety."

Classification of Bridges

The term "Classification of Bridges" is used to describe the systematic investigation of light-capacity bridges, with the view to determining the maximum loads which can safely be carried.

Formerly, the common practice, when a new engine loading was up for consideration, was to investigate all of the light bridges on the lines where the use of the heavy loading was contemplated. Stresses throughout the structures for this loading were figured, and decision then made by the one responsible for the said structures as to whether the load could be safely handled. Each time a new loading came up for consideration the process was repeated; and little or no use was made of the previous computations.

The present practice on the C. M. & St. P. Ry. is to make an investigation or "classification" of each structure. Its carrying capacity is determined in terms of a standard series of train loadings. New engine and car loadings that come up for consideration are classified in the same series of standard loadings, and it is then a matter of direct comparison to tell whether such proposed loadings can be safely handled over the various bridges. Every bridge whose date of construction indicates that it is of light design, or which is known to be, or suspected of being, overloaded, is thus classified. Every part of the structure is figured or taken into consideration.

In making these classifications it is necessary first of all to establish the maximum unit stresses to which the various materials can safely be subjected. For the different materials these maximum safe stresses are taken as near the limit of strength of the material as is considered safe. The maximum safe stresses must be assumed low enough so that there is no danger of the material yielding, altering its character, or reducing its strength for carrying loads after being subjected to this limiting stress for any number of times.

As an illustration of what may be considered as safe limiting unit stresses, the following are given, and may be taken to apply where the design and physical condition of the structure are known to be first-class:

	Pounds per Square Inch	
	Wrought Iron	Steel
Beams and girders, fiber stress in bending..............	22,000	26,000
Truss members, tension on net section..................	20,000	24,000
Timber stringers, fiber stress in bending (with suitable reduction for age in the case of exposed timber over six or eight years old)..................................	2,000	

In fixing upon limiting unit stresses for loading old bridges, it is necessary to take into account the following:

Character of design; that the details are well proportioned and direct in action, and that there is no ambiguity or uncertainty as to how the members act.

Character of the workmanship entering into the structure as indicated by the reputation of the makers and by any material-test data that may be available.

Deterioration.

Action under load, such as rigidity and freedom from excessive vibration.

Speeds likely to obtain over the structure, and confidence as to the observance of any speed restrictions that may be imposed.

Element of certainty as to the assumed loading being the maximum to which the bridge will be subjected.

Importance of traffic, and the hardship which might result thereto from temporary disablement of the structure.

Probability of early renewal on account of change of line, etc. A higher limit might be allowed for a short time to meet an emergency than would be proper for a structure to be kept in service indefinitely.

General reliability of the data upon which the investigation of the structure is based.

Generally, *judgment* founded upon all of the factors surrounding the bridge, its location, service, and condition.

It must be recognized that there is danger in setting down a hard-and-fast rule for the limits to which structures might be stressed. In all cases it is necessary to exercise care, knowledge, and good judgment, in order to be always on the safe side and at the same time conserve the maximum life of the structure.

Standard Loadings

In the systematic investigation of a large number of bridges, it is necessary to have a unit loading as a basis of comparison. The familiar Cooper's Series of Standard Train Loadings furnishes a convenient and well-known basis. This series consists of two consolidation-type engines, followed by a train load having a fixed spacing of wheels and a fixed relation between the weights on the various wheels. These weights, however, are directly proportionable to the classes; i.e., the drivers for Class E-40 Loading have 40,000 lbs. on each axle; for Class E-50 Loading, 50,000 lbs. on each axle; etc. The unit loading in this Series is taken as Class E-1 Loading.

On account of the fixed wheel rearrangement for all classes and the proportionality of wheel loads, it follows that the stresses in all parts of bridges due to these loadings are directly proportionable to the classes; that is, the stresses in every part of the structure from Class E-50 Loading will be just fifty times the stresses from Class E-1 Loading.

In connection with the live-load, it is necessary to make proper allowance for "impact," "centrifugal force," and "traction." The general method of investigating any part of a bridge and of making a classification is as follows:

1. The maximum allowable stress is determined, which, in the simpler cases, is the cross-sectional area of the member multiplied by the limiting unit stress allowed.
2. Deduct from this the total amount of stress in the part due to both "Dead Load" and "Wind Load." The remainder gives the allowable stress for the "Live Load" effect.
3. Dividing this by the stress for unit "Live Load" (Class E–1) gives the classification for allowed "Live Load," if at rest.
4. Divide this classification by the term which takes into account the extra effects of the "Live Load," due to Impact and Centrifugal Force, and the result will be the classification of the allowed "Live Load" at full speed.

Classification of Loadings

The "Class E" loading above described is an assumed typical loading. Actual engine and car loadings vary a great deal as to spacing of the wheels and the distribution of the weight on the various axles. The effects of different loadings on bridges are not in direct proportion to the weight of the engine or cars, but depend on the number of wheels, spacing of wheels, distribution of weight, etc. Actual engine loadings can, however, be reduced to equivalents in the standard train loadings, corresponding to the different span lengths.

This is done by computing the maximum bending moments and end shears for the given train loadings for each different span length. These are divided by the maximum bending moments and end shears for the Unit Class "E–1" loading, for the corresponding span, the result being the "Classification" of the loading.

As an illustration of classification of various engine loadings, Fig. 41*a* is given. This shows the classification of several types of the new standard locomotives which have been purchased by the Government and are now being assigned to the various railroads. Fig. 41*b* shows similar classifications for typical car loadings.

In placing restrictions on the use of car loadings over bridges, it is not practicable to take into account all of the variations in car lengths which occur. An equivalent for the various car loadings can be arrived at by considering typical hopper cars about 33 ft. long, as shown in Fig. 41*b*, for which the wheel spacing given is a fair average. For bridges under 50-ft. span, the trucks of two adjacent cars produce the maximum effects, and, for like axle loads, are independent of the lengths of the cars.

The approximately parallel curves on the diagram represent the classification of these typical car loadings for different weights of cars, where the total weight represents the weight of the car and contents. The said diagram shows by dotted line the classification of a typical ore-car loading, which, on account of the extremely short length of the car, produces a high classification on the long spans; and it records also the classification of a heavy wrecking crane of 120 tons capacity.

Speed Restrictions

In the foregoing the classification has been determined with an allowance for the effect of the maximum speed over bridges.

Where speed is reduced, the effects of the live load are much less; and the allowance for impact and centrifugal force, if any, may be correspondingly reduced. This will, of course, permit heavier loadings to be operated at reduced speed as compared with those permissible for full speed.

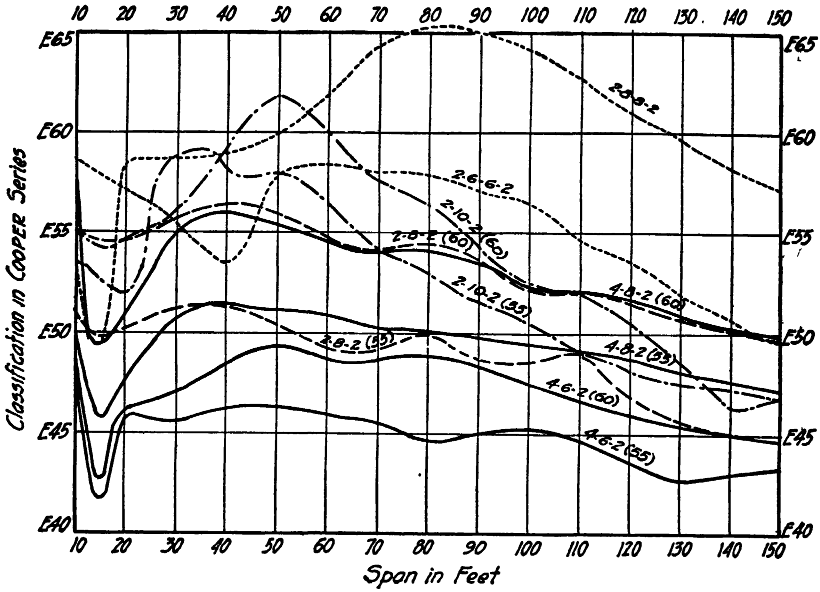

Fig. 41*a*. Diagram Showing Classification of U. S. Government, Standard Locomotives.

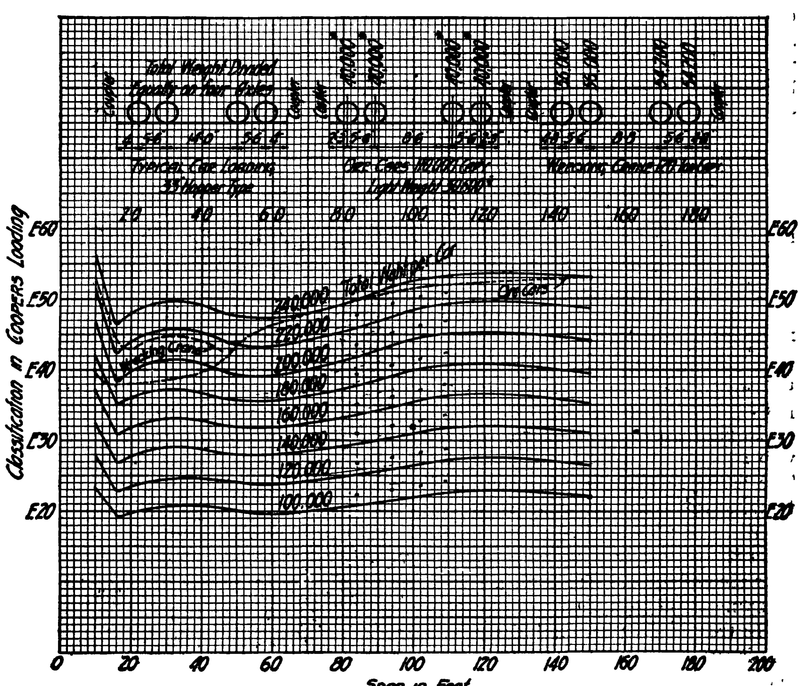

Fig. 41*b*. Diagram Showing Classification of Typical Loadings.

From the tests conducted by the American Railway Engineering Association, it is found that the maximum impact which will be obtained at reduced speed is:

Less than 30% for speed of 10 miles per hour.

Less than 40% for speed of 15 miles per hour.

Less than 50% for speed of 20 miles per hour.

Less than 55% for speed of 25 miles per hour.

An inspection of the diagrams indicates that the effective span of the bridges and the characteristics of the engine loadings determine to a great extent whether a given loading can be run over the bridge, and shows that it is unsafe to attempt to decide whether any engine loading can be handled over a structure simply by knowing its total weight.

There is, unfortunately, a misunderstanding, among some railroad operating officials, as to the effect on bridge structures of such complex loadings as locomotives and cars. In these cases it is assumed that the effect is the same for all locomotives of the same total weight; and bridges are classified as being safe, or otherwise, for all locomotives of given total weights. If this practice must be resorted to, the limits set should be on a very conservative basis; for otherwise there would be danger of certain types of locomotives having a serious effect on some bridges, producing unsafe conditions. The practice would not be economical, because it would either lead to the premature renewal of some bridges or to an unnecessary ruling off of certain types of engines.

Where Low Classification Usually Occurs in Bridges

In older bridges there are certain parts where low classifications can usually be expected. These have been found to occur most often in the lightest members of the structure and in members which carry the smallest dead-load stresses. In proportioning a member, a part of the sectional area thereof can be taken as carrying dead-load stress and the remainder live-load stress. As the dead-load stress is constant, a smaller area would be required where a higher unit stress is used. This, therefore, leaves a portion of the area originally provided for dead-load stress available to carry live-load stress.

It is found that the floor systems of bridges have generally a lower classification than the girders or chords of the trusses. The low classification of stringers is generally in the section of the flanges near the center, in the riveting of flanges near the ends (particularly if they are shallow), and in the riveting connecting the stringers to the floor beams.

Floor beams, if of shallow depth, frequently show a low classification in flanges near the stringer connections, also in the riveting of flanges near the ends, and in splices connecting the webs of the floor beams to the gusset plates, particularly in types where the lower part of the floor beam is cut out to fit around the ends of the trusses or over pins.

In plate girders, the flanges frequently show low classification at points where the web is not fully spliced near the center and at points near the ends of cover plates. The flange riveting near the ends of girders frequently has a low classification, particularly where the girders are shallower at the ends.

Webs of plate girders show low classification near the ends of the girders where there is a relatively large expanse of web, unsupported by stiffeners. The web splices near the ends of the span have a low classification where only one line of rivets is used on each side of the splice.

In trusses, the posts and diagonals near the center of the span usually show a low classification. This is particularly true of the diagonals and counter-diagonals of light eye-bars or loop rods.

Suspenders, or hip-vertical members, frequently have a low classification. The

classification of end posts and top chords of truss bridges is frequently low on account of the eccentricity of the member with respect to the location of the pin.

The pins of old truss bridges frequently show a startlingly low classification where computations are made in accordance with the usual methods; hence it is necessary to take advantage of certain conditions which are more favorable than the usual assumption, in order to help out the classification. Where eye-bar members consisting of more than two pairs of eye-bars meet on a pin, a slight redistribution of stress in the several eye-bars will frequently increase the classification of the pin; and this is justifiable as being in line with the way the structure actually works. Where certain members have wide bearing surfaces on the pins, the center of pressure can be taken near one edge of the bearing surface, thus increasing the classification of the pin and, at the same time, approximating more nearly to the actual behavior of the detail. It is also permissible to use higher unit stress for figuring pins than for the other members of the structure. The following illustrates what might be considered permissible, providing there is assurance that the material is of good quality and that the computations take account of all the forces acting:

Wrought iron		40,000 lbs. per sq. in. in bending
Soft steel	(.1%C)	45,000 " " " "
Structural steel	(2%C)	48,000 " " " "
Mild "	(25%C)	52,000 " " " "
Medium "	(.35%C)	56,000 " " " "
Hard "	(.45%C)	64,000 " " " "

It is to be noted that, in bridges built in the late 80's and early 90's, hard grades of steel were frequently used for the pins.

In timber-trestle bridges, the stringers in bending usually show low classification. On account of there being three or more sticks acting together, it is permissible to use a higher unit stress for trestle stringers than for a single stick, as the average strength for the several pieces exceeds that of the poorest one. On account of the exposure to the weather and the deterioration which gradually takes place, the allowed unit stress in timber stringers should be reduced as the age of the bridge increases. Where timber bridges are thoroughly inspected and defective material is promptly replaced, and where they are subject to the same general consideration as given above for metal bridges, the following unit stresses might be taken as a safe practice for maximum fiber stress in stringer bridges without allowance for impact:

For stringer bridges six years old, 2,000 lbs. per sq. in., and reduced about 100 lbs. per sq. in. for each year following.

The above figures are based on Douglas Fir or dense yellow pine and for climatic conditions prevailing in the North Central States. In more arid regions where longer life of timber may be expected, the reduction in stress for age need not be so rapid. On account of the comparatively short life of timber bridges and the ease with which they can be renewed, there is not generally the same urgency in establishing maximum-safe-stress limits as in the case of the more permanent metal bridges. In timber truss-bridges the lowest classification usually occurs in the floor beams, truss rods, and diagonal braces.

It has been found that metal bridges suffer frequently from corrosion in the top flanges of stringers and floor beams, on account of the action of brine drippings from refrigerator cars.

In bridges where the ties are supported on shelf-angles riveted to the webs of the girders, the shelf-angles frequently show considerable corrosion and tend to break in the root of the angle.

In pin-connected trusses, excessive wear sometimes takes place in the pin bearings, particularly in draw bridges.

Metal bridges and viaducts over railroad tracks frequently show excessive corrosion in the floor system and laterals, due to smoke and gas from locomotives, also because of the fact that the solid floors of such bridges do not permit the steel work beneath to dry out quickly.

Metallic overhead bridges having a scant clearance, so that the stacks of the locomotives come close to the steelwork, frequently show excessive wear from the sand-blasting effect of cinders issuing from the engine exhaust, particularly when the location is on an ascending grade where the locomotive is worked hard under the bridge.

Possible deterioration of the structure of the metal itself, by fatigue, has in some quarters been a matter of apprehension; but it now seems to be recognized that no such internal deteriorating action takes place where the bridge has not been subjected to excessively high stress. If crystallization is found in the metal of a structure, it probably was there at the time the structure was built, and is due to improper methods of manufacture of the material.

It may, therefore, be taken as a certainty that iron and steel bridges, if not reduced in section by rust, etc., and if not shaky on account of inadequate bracing, are fully capable of carrying the figured loads at reasonable limiting unit stresses, provided they are carefully inspected and properly maintained.

STRENGTHENING OF LIGHT BRIDGES

Strengthening of light bridges may be either a matter of reinforcing minor details which are found to limit the carrying capacity of the structures, or may consist of heavy reinforcing in an attempt to increase the strength thereof throughout. The minor strengthening can usually be done at small expense; and it is an economical method of getting considerably greater life out of bridges. Heavy reinforcing may or may not be an economical proposition, as it involves work being done in the field, which is costly, and the maintenance of traffic during the time the work is in progress, which involves some risk to traffic and is usually expensive. On very large bridges where the cost of replacing is great, some extensive strengthening operations have been carried out economically.

In making plans for reinforcing bridges, it is usually preferable to add new material to the structure so that the present structure is not temporarily weakened, rather than to remove parts and substitute heavier ones, though the latter expedient has sometimes to be resorted to. The descriptions of the points at which low classification usually occurs suggest in themselves how these might be strengthened.

In plate girders the top and bottom flanges may be reinforced by additional cover plates, particularly at points where the web is spliced and not effective for carrying its proportion of the bending stress. Where there are no cover plates on the girders, cover plates of desired length can be added. On plate girders where there are two or more cover plates, additional cover plates would be nearly the full length of the girder and expensive to apply. Plate girders can be doubled up to make deck spans, using three or more girders per span at small expense, thereby using up light girders and providing bridges of large carrying capacity.

Where waterways or other under-crossing conditions permit, timber bents can be placed under spans to strengthen them.

Where the rivets in the flanges of girders show low classification. larger rivets can be substituted for existing rivets, or, where the rivet spacing permits, additional rivets can be driven.

Where the web plates give a low classification, additional stiffeners can be placed in the panels near the ends of the girders to provide extra support for the web and thereby increase its classification.

Shelf-angles can be strengthened by placing short vertical stiffeners beneath them. Where web splices with low classification occur, these can be replaced with wider splice plates having additional rows of rivets in the splice.

In through bridges the stringers can be reinforced by additional riveting, by the placing of additional stringers, either timber or steel, and by shifting existing stringers to secure better distribution of the load. Where stringers are spaced so that some of them do not carry their full proportion of load, it is possible to introduce cross bracing so that all the stringers in the panel shall act together to carry the total load and thus relieve the excessive burden on certain stringers.

Floor beams can be reinforced by cover plates or angles added to the flanges, by additional riveting, or by shifting the stringers toward the trusses so as to reduce the bending in the beams.

In very old bridges the floor beams are frequently of much lower classification than the remainder of the bridge; and they can sometimes be replaced with entirely new beams at a reasonable expense so as to get additional life out of the rest of the structure.

In trusses, diagonals and counters can usually be reinforced with additional bars or rods having loops over the truss pins and being connected by turn-buckles to provide adjustment. Similarly, bottom chords of eye-bars can be reinforced with additional bars having yokes bearing on the heads of the original eye-bars.

End posts of through bridges, whose low classification is due to eccentricity of members, can be strengthened by placing angles or plates on the sides of the said members, so as to make the cross-sections better balanced, thus reducing eccentricity.

The bottom chords of truss spans may be reinforced by adding an auxiliary bottom chord above the present chord and sloping the end panels to meet as nearly concentric with the end pins as conditions will permit. Auxiliary web members may be connected to the new auxiliary bottom chord and the top chord. This is the method employed on the North Halsted Street Viaduct over the C. M. & St. P. Ry. Co.'s tracks in Chicago. This method has also been employed by the City of Chicago on several bridges in that city.

Where pins have low classification, it is sometimes possible to move the members on the pin and reduce the bending. In some cases, diaphragms placed in built-up members will relieve the bending on the pins. The pins themselves can be strengthened by replacing them with high-carbon or special-alloy steel pins of the same size; or, if still more strength is required, by boring out the pin holes and putting in larger pins. This operation has been done a number of times; but it requires rather elaborate arrangements for holding the members in position while the pins are removed.

Timber truss-bridges can be strengthened by placing floor beams, diagonal braces, or truss rods where needed.

Where timber trusses are old and have commenced to open slightly in the joints or to show other signs of diminished strength, they can be temporarily strengthened and carried for a few more years by placing timber bents under the panels points at two or three panels from the end of the span. This has the effect of reducing the span-length and stiffening the structure.

Timber-trestle bridges can be readily strengthened by additional stringers.

The cost of strengthening bridges varies with the size of the job, the amount of staging required, the amount of moving the same about so as to reach different portions of the work, the size of the crew available, the distance traveled by the crew, the tools at hand, etc. In a general way, it has been found that the cutting out and replacing of rivets on ordinary strengthening jobs costs from 25c. to 75c. each. Drilling new holes and driving new rivets therein cost from 50c. to $1.00 each; i.e., the cost of such work will be given by the total number of rivets driven at these unit prices, plus the cost of the additional material required.

With the maintenance of old and light-capacity bridges, the question continually arises whether it is more economical to strengthen the structure or to renew it. As a general proposition, it would be permissible to spend each year for strengthening an amount equal to the interest on the investment in a new bridge, less the cost of addi-

tional maintenance required by the old bridge on account of the greater attention it receives.

For illustration, let us consider a few lengths of through spans designed for E–55 loading, replacing similar spans designed in the early 90's. New steel work taken at 5c. per pound erected; falsework at $10.00 per lineal foot; removing old structure at $10.00 per ton; salvage on old spans at 2½c. per pound; additional cost of maintenance of the old span on account of additional inspection, classification, and supervision required, $1.00 per foot of span per year. The last column of the following table shows the amount which we could afford to spend per year in strengthening old spans rather than to renew them. The costs shown in this table are for illustration only. As they fluctuate from time to time, the resulting economics will vary accordingly:

TABLE 41a

Span	New Steel Weight	Cost Erected	Salvage	Net	Interest on Net Cost at 5%	Available for Strengthening Each Year
50′	66,000 lbs.	$ 4,330	$ 1,130	$ 3,200	$ 160	$ 110
100′	218,000	13,390	3,700	9,690	485	385
200′	800,000	46,000	12,500	34,500	1,725	1,525
300′	1,600,000	92,200	25,000	67,200	3,360	3,060

The writer has in mind a bridge having three 400 ft. spans which, if renewed about ten years ago, as some Railroad Managements might have done, would have cost about $370,000 after deducting the salvage value of old spans recovered. The interest on this investment for the ten years would have amounted to about $185,000. Instead, however, of replacing these spans, they have been carefully maintained and inspected and the details strengthened wherever the classification showed that it was necessary to carry the heavier traffic. The actual cost of strengthening, together with the additional maintenance expense, has amounted to not over $20,000 during this period, showing a saving for this bridge of about $165,000; because of the policy of getting the longest practicable life out of structures.

This illustration is intended to show only one way in which the problem may be considered. With old and light bridges a limit is reached beyond which it is not economical to strengthen them; and replacement then becomes necessary. It must be recognized, of course, that a newly designed and heavy structure is preferable to a lighter one. It is possibly true that, in case of a serious derailment on a bridge, a light structure might be destroyed while a heavy new structure might withstand the same treatment without being seriously disabled. Such considerations must be taken into account in shaping the general policy concerning the keeping of light bridges in service.

From the data furnished by Mr. Heritage, the following has been excerpted:

Strictly speaking, the economics of maintenance and repairs starts with the design of the bridge. The most economical structure is that on which the total fixed annual charges are a minimum—these charges to include

interest on the original investment, annuity to provide for the replacement of the structure when the time comes for its renewal, insurance when necessary, and charges for maintenance and repairs.

It is evident that, by increasing the original investment, we can usually provide a structure that will have a longer life and will require less annual expenditure for maintenance and repairs. However, we are considering here only the subject of maintenance and repairs, and shall take into account only such structures as are actually built, the economics of design and the types of structure best suited for various purposes and conditions having been covered in other chapters.

All structures should have periodical and careful inspection; and when anything is found requiring attention, it should be done promptly. The neglect of attending quickly to minor repairs needed will usually result in much more extensive and costly repairs later on; for, according to an old and well known proverb, "a stitch in time saves nine."

When an organization, such as a Railway Company or a City, has a large number of bridges to maintain, it is customary to have a department to look after the work, with the requisite force of men and the necessary equipment; and these should be continually engaged in making repairs to bridges and like structures.

Repairs to timber bridges are more frequent than those on permanent structures, on account of the liability of timber to decay. Wood preservation, especially by the process of creosoting, has taken wonderful strides in the last few years. With the growing scarcity of timber and the rise in its price, such preservation will increase until it will be the exception rather than the rule to have untreated timbers in exposed structures. The majority of wooden bridges now in service on railways are trestles; although, in some parts of the country, timber or combination-timber-and-steel truss-spans are still used. Timber structures can be kept up almost indefinitely by replacing members shortly before they become dangerously decayed; however, when a bridge is so old that a large portion of the timber shows more or less decay, it is more economical to renew the whole structure, salvaging such second-hand material as possible, after which the bridge will be serviceable without further repairs for a period of years.

In case of pile-driven trestles, it is cheaper under some conditions to replace the piles with frame bents than it would be to renew the structure as a pile-driven trestle. In this case the old piles are cut off under the ground until sound timber is reached, usually about two or three feet; and the new frame bents are supported on these old piles. The latter usually rot off close to the ground line; but in good soil they will remain sound at a depth of two or three feet for a long period of years. The objections to this procedure are that the sill, being underground, is difficult to inspect and that a frame bent structure is not as stiff as a pile structure; consequently, if a pile driver is available, it is preferable to drive pile bents rather than to construct frame ones.

In making repairs and renewals to timber bridges—and, in fact, all bridges—the bridge gangs formerly were usually equipped with only hand tools; but, now that the cost of labor is so much greater, it is economical to replace manual labor as far as possible with machines. A light derrick car equipped with a hoisting engine or a locomotive crane will save the work of many men in handling heavy bridge material. On timber structures a large part of the labor consists of boring holes for bolts and fastenings. Repair gangs should be equipped with boring machines operated by either air or electricity, in order to do this work economically. The magnitude of the task will determine whether machinery or hand labor is the more economical. On very small jobs it will be more costly to set up the equipment than it would be to do the work with hand tools; however, the tendency is to make more and more use of machines instead of manual labor.

The preservation of timber, in order to increase its life and reduce the necessary maintenance, has been mentioned. Formerly timber highway bridges were commonly covered by roofs and siding to protect the frame work from the weather; and such structures lasted for a long period of years, several notable wooden bridges in this country having reached an age of nearly 100 years. This sort of bridge accumulated a great deal of dirt and was very dark at night, and the roof, housing, and floor naturally required considerable maintenance.

The question of fire protection should also receive proper attention in connection with timber bridges. The expense caused by a burned bridge in some cases far exceeds the value of the structure itself, as, for instance, on a railway where the traffic is stopped on this account. On railways, timber trestles are frequently partially protected from fire by covering the deck with sheet metal or with stone or gravel, so that sparks from a defective engine will not set it ablaze. Sometimes fire-proof paints are used. Some of these paints are very effective for several years, and at the same time are good timber preservers; hence, if properly selected, they will be economical from a maintenance standpoint as well as in respect to protection from fire. The timber floors on highway bridges and the ties on railway bridges are subject to wear, and are usually worn out before they rot out. In cases of very heavy traffic, bridge floors should be constructed of more permanent material than planks; and the ties on railroad bridges should be protected with tie plates.

Old stone piers and abutments often show open seams where the mortar has fallen out of the joints. If these receive attention in time, it will usually be sufficient to dig the old mortar out of the seams and repoint them, thus protecting the interior of the structure from moisture; but in some cases more work than that is necessary. On high structures, especially over rivers, it is very costly to put the spans on falsework in order to repair or rebuild piers, hence other expedients are resorted to, in order to avoid this

expense. Sometimes a jacket of reinforced-concrete can be built around an old pier and the top protected with a water-proof coating.

Concrete has largely displaced stonework for the material of substructures in bridges. Piers made of iron or steel cylinders and filled with concrete provide a cheap construction where conditions are suitable. They were formerly largely used for railroad bridges, but their employment is now confined mostly to a light class of highway bridges or to railroad bridges carrying only light loading. If this type of pier is very high, it is subject to vibration under traffic; and this vibration will produce an injurious effect on the superstructure as well as on the piers. Many piers of this class are kept in service by encasing them in concrete, thus increasing their weight and stability; and this can usually be done without putting the bridge on falsework and removing the old piers, because the spans will be supported upon the old cylinders while the new work is in progress.

Steel-frame structures are subject to deterioration by corrosion; and to prevent this the surface must be always covered by a protective coating, usually paint. Since corrosion will gradually eat away metal which cannot be replaced, it is evident that this protection is of the utmost importance. In many cases it does not receive the attention it deserves. Many bridges have been seriously damaged by rust, even to the extent of having to replace them, all of which expense could have been prevented by keeping the structures properly painted. Railroads usually recognize the importance of painting, and seldom allow the rust to accumulate to the extent of weakening the structure. However, even on railroads, bridges are found that have been materially damaged by corrosion. There are parts specially subject to rust, such as the top flanges of deck girders and of stringers that are more or less hidden by the ties, and which sometimes become badly corroded while the paint on the main body of the structure is still in a good state of preservation. Also parts of highway bridges underneath the floor are frequently in bad condition, owing to the fact that they cannot be noticed by anyone crossing the bridge. However, if proper provision for inspection at regular intervals is made, there is no excuse for this condition resulting in damage to the structure.

There are many varieties of paint recommended for steel bridges. The best are usually the most expensive but will be the most economical in the end. The application of paint being about twice as costly as the material, a saving of a considerable percentage in cost of paint will result in only a small percentage of economy on the entire job. The result is that the cheaper paints will last probably two or three years less than will the more expensive ones. The subject of bridge paints is highly technical and cannot be gone into in detail here; however, it is important to note that different paints should be used for different conditions. For instance, a paint that would give satisfactory results in a dry climate would not be suited for a structure subject to acid fumes or engine blasts, as on bridges

that are over railroad tracks, or for a moist climate, especially near salt water. Some paints are a good protection against corrosion but do not stand the weather, while others have exactly the opposite properties. Therefore, the first or priming coat should be of the first-mentioned variety and the finishing coats of a paint that is not readily affected by exposure to the weather. The most commonly used and satisfactory paint for the priming coat is pure red lead ground in linseed oil. Sometimes linseed oil alone is used and sometimes paint with a Portland cement base. For the finishing coats, graphite paints, graphite and silica paints, or paints of other carbon pigments, such as lamp black with oxide of iron, or oxide of iron paints are most frequently employed. The latter are not suitable for the priming coat, owing to the fact that these pigments promote oxidation, a condition that is often ignored in practical work.

As to the application of the paint—this must be well done, if satisfactory results are expected. Metal should never be painted when it is damp, in freezing weather, or over rust. In repainting old structures probably the most important consideration is the cleaning of the metal. Paint applied over rusted surfaces will not be durable. Further, there is likelihood of corrosion spreading underneath the paint, and then the protection will soon break down. As previously noted, some parts of the structure are subject to faster deterioration of the paint than other parts. This is true of the tops of stringers, floor beams, and compression chords, and of flat surfaces exposed to weather, especially surfaces beneath the floor, such as lateral plates, battens, etc., which give lodging to cinders and other materials that will hold moisture. For this reason a very satisfactory method to use in painting an old bridge, where the paint is bad on certain parts and fair on others, is to clean thoroughly the metalwork and paint over the parts where it is exposed with a coat of red lead or some other good priming paint, and then give the whole structure a good finishing coat.

As to the method of cleaning rust from the metal—this is done by cutting with chisels and hammers and by scrubbing off with wire brushes, or with a sand blast. The latter method is most effective; however, it is expensive and is not recommended unless the bridges are in a bad state of corrosion. Also it must be used with a great deal of caution so that in removing the rust an excessive amount of the surrounding metal will not be cut away at the same time.

To obtain satisfactory results, a great deal of care must be taken in applying the paint; it should be brushed out thoroughly on the metal. Recently painting by air-spraying machines has come into extensive use, but the employment of these machines on bridgework is not common. It is doubtless a labor saver and, if properly handled, is not very wasteful of paint. The manufacturers of these devices claim that the work they do is superior to hand painting; and, when skilfully used, it is certain that good results can be obtained. However, for durability, it is probable that such coatings are not as good as paint well brushed on by hand; still it might be

economical to use spraying machines on bridges, if the labor cost saved will more than offset the decreased life of the paint. Painting by spraying machines would be satisfactory on large vertical surfaces, such as the sides of girders and beams and the faces of large posts and columns; but it is not recommended for lattice work and other small parts, on account of the wasting of paint. One advantage derived from the use of these machines is the application of paint in places that are difficult to reach with a brush. On flat surfaces like walls, recent experiments indicate that the air spray will decrease the labor cost by more than one-half, with a slight increase in amount of paint used, about 5%; but similar data for bridgework are not available.

Deterioration of steel structures due to wear will be apparent in the loosening of rivets and in the increased vibration caused by cutting in pins and other parts. When loose rivets appear in the structure, they should be cut out and replaced. They are generally most frequent in lateral bracing of stringers and girders and where single-angle bottom-laterals are attached to the stringers. Also stringer and floor-beam connections are likely to develop loose rivets when the structure is overloaded.

In old and heavy bridges the pins will often show considerable wear. This is an expensive matter to repair, as it involves putting the structure on falsework and the use of considerable machinery to re-drill the holes. If pins are replaced, the new pins should be slightly larger than the original ones. This class of repair work is not often done, because a bridge in this condition is usually very much overloaded and should be replaced with a heavier structure. Another location for wear is at the intersection of diagonal bars that are in contact. In order to prevent these bars from cutting away, a buffer should be placed between them and clamped thereto.

On old bridges it is frequently found that the eye-bars making up one member are not pulling evenly. If the member consists of two bars, a satisfactory repair can be made by cutting out a piece of the loose bar, inserting a turn-buckle, and drawing it up to the same tension as that of the tight bar. This can be done without falsework, but traffic must be kept off the bridge while work is in progress. The same procedure can also be adopted for the outside bar of members composed of more than two bars, but on the inside bars it is impracticable, because rivets cannot be driven.

When truss bridges are more heavily loaded than was originally contemplated, the overstress is likely to be greatest on the counters. Also on a long span the overstress on the floor system is liable to be considerably greater than it is on the main truss members. For this reason it is often possible to make the structure safe for considerably heavier loading by reinforcing these parts and keeping it in service instead of replacing with a heavier bridge. Also in some cases it is possible to improve the structure without strengthening it; that is, on a very light bridge, subject to large vibrations under trainloads, it is practicable to improve the action by

putting in suitable bracing that will decrease the vibrations and the wear, although, strictly speaking, it will not strengthen the main truss members.

This dissertation does not aim to describe all the details of repairs necessary for bridges, because usually special problems in each case are developed; but the underlying principle of economics of maintenance of bridges and similar structures is to keep a close watch on them by frequent and thorough inspection, and to repair promptly any damage that is found. The neglect of prompt attention on a structure often leads to much unnecessarily expensive work and sometimes to the development of dangerous conditions.

The joint data of Mr. Chalfant and Mr. Covell were contained in two letters from which the following extracts have been made:

The subject will be divided for convenience into three parts, viz:

A. Masonry.
B. Floors.
C. Painting.

(*A*) *Masonry*

The constant tendency of small streams to change their courses necessitates regular inspection to prevent undue scouring and cutting of the banks and approaches. Where the nature of the foundation and depth of masonry are a matter of record, the soundings and measurements can be interpreted with comparative ease, but where this knowledge is lacking, as is true in many cases, chances should not be taken. Sometimes barriers of a more or less permanent nature can be erected above the bridge in such a manner as to turn the stream back into its natural channel, or the channel itself may be changed. When this cannot be done, heavy rip-rap should be placed around the masonry in such a manner as to fill the hole and prevent further erosion. This rip-rap may be formed of rough stones from the quarry or of blocks of concrete made at the site. When concrete is used for protection, it should be in loose blocks which are free to settle, rather than in a solid mass which, in turn, may be under-scoured.

When the stream is of sufficient size to require piers in the channel, with other than rock foundations, there should be a systematic program of soundings so that the conditions of the stream-bed around the piers may be known, and so that changes may be noted. This is essential when the masonry rests on piles, but is of even greater importance when the foundation consists of a timber grillage on gravel or other hard stratum. Heavy rip-rap is a most excellent protection to piers against scouring of the bed of the stream; but, when once placed, there can be no assurance of future security. The rip-rap may be moved by ice so that the bed around the pier is again exposed to erosion.

(B) Floors

When the strength of the superstructure will admit of such loading, the roadway floor should be reconstructed with a reinforced-concrete slab and brick wearing surface, and the sidewalk should be of reinforced-concrete. In some cases, where new steel stringers are required, buckle plates may be used with some form of comparatively-permanent wearing-surface. Where the foot traffic is heavy, the sidewalk can be given an asphalt wearing-surface. Few of the older bridges are heavy enough for the loading indicated, having been originally floored with plank. The time for use of a timber floor with the side of the grain exposed to wear has passed in most places. The spikes work up and cut automobile tires, and the floor soon requires renewal. Lumber is now so costly that it is not usually economical to employ untreated timber in so exposed a place as a bridge floor, and the use of treated lumber only, with wood-block wearing-surface, is recommended.

We have scores of iron and steel bridges over rivers and small streams, which structures are too light to carry a concrete and brick roadway floor or a concrete sidewalk. The weight of trucks now traversing these bridges makes the use of wooden stringers very undesirable because of the presence of knots and other defects, hence steel stringers are employed. The practice in some places is to lay planks directly on the steel stringers with only occasional fastenings, but this does not seem to be satisfactory. Bolting at each bearing point is expensive, hence our practice is to bolt a surfaced $2\frac{3}{4}'' \times 5''$ nailing piece on top of each stringer, nailing the planks with two nails at each intersection, as in the case of wooden stringers. It has been found that such solid nailing distributes the load so that a wheel load is carried by at least two stringers. On such bridges the stringers should be from 24″ to 28″ center to center; and planks of uniform width, at least 10 inches wide and surfaced to $2\frac{3}{4}$ inches, should be laid parallel with the backwalls, even though the skew necessitates extra-long planks. This prevents pointed ends, which are hard to support. In extreme cases the planks may be in more than one length across the floor, but the joints in adjacent planks should not come over the same stringer or adjacent stringers, but should lap for a distance equal to at least two spaces between stringers.

In case the bridge carries street-car traffic, there seems to be no better construction than to employ ties to support the rails and planks. Seven-inch grooved rails and half-inch tie plates are used. Planks at least 10 inches wide and surfaced to $3\frac{3}{4}$ inches give good results. The blocks should then have sufficient depth to come flush with the rail. A shaped wood-filler on each side of the rail is much lighter and more permanent than concrete filling. It is very important, however, that each tie shall have but two bearings and that the rails be as nearly over the stringers as possible, otherwise the springing and warping of the ties will give an irregular bearing for the rails, resulting in future trouble. In this connection it might be suggested that all street-car rails be painted, when laid, like all other struc-

tural steel, omitting paint from the head if desired. The rails, in many installations where the traffic is comparatively light, rust out in the web and base, rather than wear out.

Wood blocks may be laid directly on the planks, but the usual practice is to place a single layer of tar paper between, so as to prevent loss of the hot joint-filler. A very convenient depth for the wood blocks is 3½ inches, but, whatever depth is selected, there should be at least one-fourth inch difference between the depth and the width, so that they will not be laid accidentally with the side of the grain up. On long bridges, especially where there is a grade, angles should be fastened to the floor with lag screws at intervals varying from 10 feet to 30 feet, according to conditions, in order to prevent the blocks from creeping. On grades greater than two per cent, "hillside" blocks, made like hillside brick, can be used to good advantage. We have had a short section of such floor in place on an eight per cent grade for several years with good success. Where there are no street-car rails to prevent side drainage, it is well to give the roadway a crown of two or three inches in order to assist in draining.

While it is true that the floors described are heavier than the plank floors which they replace, this is offset by the comparative smoothness of the surface, which reduces the vibration caused by passing vehicles.

The sidewalks on our lighter bridges have been constructed with either treated or untreated lumber laid in the usual manner, but this is not very satisfactory. Such floors wear out rapidly, if the traffic is heavy, and are rough and irregular. Treated lumber exposed to wear on the side of the grain does not last well and does not make a good sidewalk. We have recently laid a sidewalk on a river bridge with tongued-and-grooved lumber surfaced to 1¾ inches, and with wood blocks 2 inches deep, 3 inches wide, and averaging 6 inches long, laid thereon. Each block in every fifth row was nailed with a 10*d* wire finishing nail to prevent movement or displacement, and the joints were filled with dry sand. It was found, while the work was in progress, that blocks of this depth, on a surface where there was no wheel traffic, remained loose so that mischief-loving boys lifted them out of the walk and threw them into the river. The joints were then filled with hot bituminous filler, as was done on the roadway, but care was taken to cover the surface immediately with dry sand before the filler had time to cool. The resulting surface is very satisfactory.

When the old plank floors on the smaller bridges are replaced with the wood-block floor as described, the grade is usually raised a few inches. This necessitates an increased height in the back-wall. At first this was accomplished by taking up the sand-stone back-walls and resetting them at sufficient height to dress to the new floor-level. This is expensive and not wholly satisfactory; and for three years the same result has been reached by cutting the old back-wall down, where necessary, and setting two rows of paving brick at right angles to the face of the said back-wall in a bed of Portland cement mortar, and grouted in place. Where the approaches

have an earth wearing surface they are raised by the use of broken stone; but where they are improved with macadam, bituminous surfaces, or brick, a brick surface is used. Bituminous concrete by the penetration method and Portland cement concrete have both been employed; but under the conditions prevailing here, neither has proved satisfactory. The former requires a heavy roller to give good results, and the size of the contract will not warrant this; and the latter should be kept free from travel for a longer period than is required to complete all the rest of the job. Our practice on small bridges, in all cases where the traffic cannot be readily diverted, is to cut the old floor in half and put in the new stringers and nailing pieces on one side. A temporary floor is then laid on this side, and the stringers and nailing pieces are put in on the other side. The new floor can then be laid without serious interruption to vehicle-traffic. Foot-traffic is maintained continuously. We use $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$-angle wheel-guards 10 inches above the floor in all through, plate-girder bridges in place of wooden wheel-guards.

A complete itemized record of the repairs on each floor should be kept in the office. A brief summary of this, giving the date and extent of the repairs, should be prepared and placed in the hands of the inspector for the annual inspection in the spring or early summer. This is especially important in the case of ordinary plank floors. When the inspection is made, any accumulation of mud or dirt should be cleaned from the floor, which should then be carefully examined for needed repairs. Ordinarily it is difficult to detect decay in the top of the stringers, until the hard, sound shell on the outside breaks down. If the inspector has the date when the stringers were put in, his experience will tell him when he needs to expect this kind of failure; and planks should then be taken up to make sure.

Records kept in this manner enable the person in charge to have definite knowledge of the conditions in the field; and an inspection made with such information at hand forestalls many floor failures which otherwise frequently occur.

(*C*) *Painting*

We have found that our painting is best done by day labor with paint furnished by the owner. It frequently happens that parts of the bridge need to be only touched up at various points for the first coat, in order to give a uniform surface over the whole bridge; and the extent of such work cannot well be specified in advance. Cleaning is fully as important as the painting; in fact paint applied over rust, scale, or dirt is worse than wasted, for it may prevent the real conditions from showing and thus foster further corrosion. With a good, conscientious foreman, the cleaning can be made more thorough than is usually possible under any inspector with contract work. At this point it might be well to observe that many detailers seem to forget all about maintenance, and especially about cleaning and painting, when the drawings are made. Clearances are too small, and exposed metal

is placed in inaccessible places. This can sometimes be corrected with concrete.

A painting program would include the painting of small bridges in scattered positions every three or four years. If the territory is extensive, it should be divided into sections, one of which should be included each year. While there are many good proprietary paints, there is no very satisfactory way of selecting them on a competitive basis; and we have found it better to purchase the paint under specifications upon which all manufacturers can bid.

Dark paints seem to be more durable than light paints; and bridges which are lighted at night can be painted black or some other dark color. Bridges which are not artifically lighted at night should be painted a light color to make them more readily visible. Rather than paint the entire bridge a light color, the floor system and all parts below the usual line of vision (parts where it is generally most difficult to maintain paint) may be painted dark. When a new floor is put on, the tops of all floor beams, sidewalk brackets, and stringers, and other parts ordinarily inaccessible but made accessible during this work, should be thoroughly cleaned and painted.* This sometimes involves a hardship by causing delay in the work when traffic is maintained; therefore, instead of two coats of paint, a heavy coat of red lead paste can be used. If creosoted lumber is to come in contact with steel, the ordinary paint will not stand, as creosote is a solvent; hence the finishing coat should be a specially-prepared, acid-proof paint.

Where the finish is dark, the priming coat may be red lead and linseed oil paint; and where the finish is light, a good priming-coat pigment is made of 29% lead sulphate, 41% lead carbonate, 5% zinc chromate, 10% silica, and 15% asbestine. A good black finishing-coat pigment is made of 55% lamp-black and 45% special French ochre; and a finishing-light-coat pigment can be made of 34% lead sulphate, 41% lead carbonate, 10% silica, and 15% asbestine, with sufficient lamp-black to produce a pearl-gray color. This is light enough to be readily seen and stands better than pure white. The vehicle should be pure linseed oil with the necessary drier. All proportioning of ingredients should be by weight.

Where possible, all steelwork directly over steam-railroad tracks should be protected by concrete rather than by paint. Where this is not practicable, the painting should be done at more frequent intervals than is ordinarily necessary.

The preceding records of the opinions of four engineers who are experts on maintenance and repairs ought to afford the reader sufficient data on the subject to serve all practical purposes. There is some unavoidable repetition involved, and there are some minor differences of opinion, because the economics of maintenance and repairs is far from being an exact science. Again, the treatment of the matter of painting encroaches on the special

subject of the next chapter; nevertheless, it was not considered advisable to omit any salient portions of the three dissertations. A little repetition in a technical work does no harm, when it expresses the opinions of several authorities; and the different points of view recorded are certainly both interesting and advantageous.

CHAPTER XLII

ECONOMICS OF METAL PROTECTION*

THE preservation of bridges against rapid deterioration is just as important a matter as ensuring that they are properly proportioned and constructed—yes, even more important, for what behooveth it the owner of a steel structure to take the utmost care in its designing and building, if he neglect to protect it effectively against the ravages of rust? The life of a metal bridge that is scientifically designed, honestly and carefully built, and not seriously overloaded, if properly maintained, is indefinitely long, but if badly neglected is often quite short, especially when it is exposed to acid fumes, such as those contained in the smoke from locomotives passing through or beneath. It is evident, therefore, that the subject of economics of metal protection is one of consequence and deserving of the most thorough consideration.

It may appropriately be divided into two topics, viz., the general question of economic expediency in guarding the structure against injury by the expenditure of considerable money, and the cheapest ways of effecting satisfactory protection.

The first topic may readily be disposed of by the statement that it is in the line of true economy to spend whatever amount of money (within, of course, the bounds of reason) that is found to be necessary to prevent the starting of any rusting of metal whatsoever. If, as many people think, the life of a steel bridge is limited to two or three decades, the economic question would arise as to how much money it would pay to spend on painting or other protection, in order to prolong the said life a few years; but such is by no means the case, because, as previously indicated, a modern steel bridge ought to last for centuries.

The second topic covers a wide field, and requires to be treated in detail. *The principal subjects that it includes are the following:*

1. Best kinds of paint to use in shop and field.
2. *Best vehicle for pigments.*
3. *Use of driers.*

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* After this chapter had been finished for some time it was submitted for criticism to the veteran paint specialist, Dr. A. H. Sabin, Consulting Chemist to the National Lead Company, and the acknowledged dean of American paint men. Dr. Sabin very kindly prepared some memoranda on certain points; and, in order not to necessitate a re-writing of the entire chapter, his suggestions have been incorporated as foot-notes.

4. What colors for paint are best suited to different conditions.
5. Elasticity of paint coats.
6. Covering and spreading powers of paints.
7. Cement paints.
8. Linseed oil alone for the shop coat.
9. Climatic influences on paints.
10. Application of paint by spraying.
11. How best to prepare new metalwork to receive the shop coat.
12. Pickling.
13. How best to paint newly-erected metal.
14. Concrete encasement.
15. Gunite.
16. Proper treatment of steel that is to be encased in concrete or gunite.
17. Water-proofing.
18. Protection of metal against brine drippings.
19. Protection of metal against locomotive gases.
20. Causes of paint deterioration.
21. How to care for incipient failure of paint.
22. How to determine when repainting is necessary.
23. How to clean the metalwork preparatory to applying a new coat of paint.
24. Application of paint after field cleaning.
25. Factors that affect results in painting.
26. Economic observations concerning painting in general.

The topics in the above list will be taken one at a time and discussed from the economic view-point.

Best Kinds of Paint for Shop and Field

Concerning the best kinds of paint for bridges there has been waged a lively war of competitors during half a century or longer, each one claiming that his product is the best. Independent engineers, too, have varied in their views thereon, for each one has been rather prone to be influenced by his own personal experience; but of late years a general consensus of opinions has been reached, the decision being that the priming or shop coat should be red-lead paint, the first field coat a mixture of red-lead and some so-called inert material, and the third coat a carbon or graphite paint. The term "inert" as applied to paint constituents was originated some thirty years ago by the late Dr. Dudley. It reflects his idea that lead and zinc pigments are chemically active towards linseed oil, while barytes, silica, etc., are not. As a matter of fact, the most important, if not the only, relations between the pigment and the vehicle are physical, and in that sense there are no inert pigments. Nothing can be less chem-

ically active than powdered quartz, yet it will bleach oil as thoroughly as is possible in any way.

Some valuable suggestions concerning the composition of shop and field coats are given at the end of Chapter XLI in the data supplied by Messrs. Chalfant and Covell.

There seems to exist among engineers a notion that it is unprofessional for a technical writer to recommend in print any special make of paint. Such a tenet, however, is fundamentally wrong, as is also the idea that one should not call for any particular material of any kind in his specifications. If an engineer is confident that a certain material will suit his proposed construction better than any other, he should have the courage of his conviction and should call for its use, even if he has to run the risk of evil-minded persons insinuating that he was illegitimately influenced so to do. Similarly, when a technical writer has learned from long experience that certain materials are best for certain purposes, he should be sufficiently brave and independent to give to his brother engineers the benefit of his accrued knowledge.

For many years the author has favored red lead for the shop coat, provided that it were honestly manufactured, honestly mixed, honestly applied, and honestly dried before being either covered or subjected to possible abrasion. Again, experience has taught him that the pigment should not be delivered at the shops as a powder or even as a paste, but that the paint should be previously mixed and ready for use; because the caliber of the men employed for shop-painting is generally so small that they cannot be trusted properly to mix the paint, hence it has usually occurred that the mixture was lumpy, that thinners were illegitimately added, and that the coating was daubed on the steel irregularly, being too thick in some places and too attenuated in others.

Year after year the author has continued his search for an ideal shop coat; and it was not until a short time ago that he found it in Dutch Boy Red Lead. At first he could obtain this only in either powder or paste form, with which it was impracticable to obtain perfect results; but finally he persuaded the manufacturers to furnish it ready mixed, using to one American gallon of pure raw linseed oil twenty-eight pounds of the pigment thoroughly incorporated by grinding with the oil. He is now satisfied that he has discovered what he has been searching for during more than three decades. For the field coat he has had the most satisfactory results with Goheen's Carbonizing Coating, Nobrac, and Detroit Graphite, although at one time long ago, as will be explained further on, the latter paint failed him.

Other engineers have had good luck with other finishing coats—for instance, those recommended by Cheesman & Elliot, Lowe Brothers' "Metalcote," and Toch Brothers' "Tockolith." Lowe Brothers' "Red Lead Lute" has given good satisfaction as a shop coat; and for that purpose many users pin their faith on Cheesman & Elliot's No. 31 Red Oxide. A

quarter of a century ago the author used to employ the last-mentioned paint as a finishing coat, and found it excellent; but for a shop coat he believes that nothing is as good as truly-first-class, red-lead paints.

Between 1906 and 1912 there was made, on the Havre de Grace, Md., bridge of the Pennsylvania Railroad Company, an elaborate and exceedingly valuable series of tests of nineteen different kinds of paint under the direction of certain expert engineers. The results, as reported, emphasize the fact that red-lead paints and those having a considerable amount of red lead in their composition are the most durable, and that a paint in which the top coat is strictly a preservative cover over the red-lead coat gives the best results of all, and justifies the well-known philosophy of the late Mr. Houston Lowe regarding protective paints for steel.

To quote the exact words of Mr. Geo. S. Rice, one of the engineers who reported on the said tests, "this philosophy prescribes the production of a solid and sufficiently-elastic foundation with rust-restraining properties by a priming coat of red lead, followed by a transition coat intermediate between the primer and the top coat, the office of the latter being essentially protective of the undercoats."

It was noticeable in the test that the paints which withstood best had in all instances a very large percentage of pigment in their composition.

The winning paint in the competition was a combination of the kind just described, submitted by the Lowe Brothers Company.*

In his book "Paint for Steel Structures" Mr. Lowe expresses these conclusions:

1. That the priming, or first coat of paint, upon any surface is the most important one; and that it should form an inhibitive, firm, unyielding, and receptive foundation for those to follow it.
2. That under-coats should dry harder and more quickly than those above them, and that the difference in drying between adjoining coats should not be very great.
3. That the quality of the binder is equally as important as the quality of the pigment.
4. That the quantity or weight of pigment used is equally as important as its quality or volume.
5. That the time and method of application are equally as important as the quality of the paint.

There used to be some well-founded objections to red-lead paints in general, and these still hold good against the cheap varieties thereof. The principal ones were a tendency to sag and run on vertical surfaces, and

* This statement is on the authority of Lowe Brothers' written claim, but it has been contradicted by Dr. Sabin, who says that the winning paint was a pure red lead manufactured by the National Lead Company. Dr. Sabin prefers to use red-lead paint for all three coats instead of for the shop coat only, in spite of the very prevalent idea that red lead, for efficiency, should be covered by a more elastic pigment.

to settle into a hard mass in the bottom of the container. These faults were due to an excessive amount of litharge in the pigment, sometimes as much as thirty per cent. Within a few years certain lead-paint manufacturers have reduced the litharge to as low as two per cent, the remaining ninety-eight per cent being true red lead, Pb_3O_4. This makes an ideal paint for the priming coat; for, being extremely fine, it fills all pores, and brushes out in a smooth, even film free from voids. Moreover, it stays in place on vertical surfaces, does not act ropy under the brush, and does not settle to the bottom of the container. It is sold generally in paste form; but, until it can be regularly furnished ready-mixed for application, it will not have attained its acme of excellence.

The amount of red lead to be used per American gallon of vehicle is still a disputed point among engineers. In some cases the amount actually employed has been as high as thirty-seven pounds; but such an unusually great quantity cannot be made to give satisfactory results, unless all the conditions are ideal. If the paint be applied under contract, which is by no means the best way but sometimes is unavoidable, it is well to limit the amount of pigment to twenty-eight, or possibly thirty, pounds per gallon of oil.

The theory one should adopt when applying the coats which follow the priming coat, as well as at any time thereafter when the bridge is to be repainted, is to have each coat more elastic than the one preceding it, so as to insure against checking and alligatoring—a term very aptly applied to what occurs when paint dries in lumps or ridges or when it shows wide, irregular cracks, giving the surface an appearance of alligator hide.

Some authorities advise adding a little non-drying oil to the final coat of paint, in order to enable it better to shed water; and the author agrees with this practice, provided that the amount used be not great enough to prevent the paint from drying thoroughly by the time an additional coat is required.

Summing up the matter of the best kinds of paint for bridgework, the author feels that he cannot do better than to quote the following from Mr. Houston Lowe's "Paints for Steel Structures" concerning the desirable features of an anti-corrosive metal coating:

1. It should hide the surface.
2. Should cement itself together, and also cement itself to either damp or dry metallic surfaces.
3. Should expand and contract without breaking its own body.
4. Should present a hard, yet tough, outer surface.
5. Should be impervious to water, carbonic acid, or other gases.
6. Should be unaffected by sunshine, heat, frost, dew, or climatic changes.
7. Should be unaffected by ordinary mechanical abrasion.
8. Should wear evenly.

9. Should fail by gradual wear, not by disintegration.
10. Should leave a good surface for repainting.
11. Should not require an unreasonable amount of skill or muscle in application.
12. Should be homogeneous.
13. Should dry fast enough.
14. Should not be readily ignited.
15. Should have power to absorb and remove moisture or dampness from the metal.
16. Should have properties that will prevent corrosive action of traces of water in contact with the metal.

Best Vehicle for Paint

Durability of paint depends just as much upon the vehicle as it does upon the pigment. Up to the present time no vehicle has proved to be anything like as good as pure, raw, linseed oil, notwithstanding the fact that many substitutes have been tried. Some of these substitutes are valuable as thinners of linseed oil, if used in moderate quantity, because they often improve somewhat both its drying and its working properties. The usual reason, however, for the adoption of such thinners or adulterants is to reduce the cost of the paint; and too often this is done at the expense of its quality. It is generally conceded that "the base of the best substitutes for linseed oil is linseed oil itself."

The author once had great hope of Leucol Oil as a vehicle; and, as a test of it, he used Leucol Red-Lead Paint on one of his Mexican bridges in competition with a number of other paints on several nearby structures; but it failed to give satisfactory results, the surface quickly assuming a whitish tinge, and the protection failing much sooner than it should have. It should be stated, though, that the climatic conditions in the *tierra caliente* where these structures were located were unusually severe for bridge paints. All but one of the paints then tested failed unequivocally—but of that exception, more anon.

Boiled linseed oil was much used for bridge paint in times long past; but experience has shown that the boiling is a detriment to the vehicle instead of an improvement thereto.

Use of Driers

Some engineers consider all driers merely as adulterants, employed solely for the purpose of cheapening the product; but the author is of the opinion that they occupy a legitimate place in paint manufacture, provided they be used in moderation—especially for priming and intermediate coats, which often need to dry fairly quickly in order not to delay the application of the succeeding coat. As far as he knows, the best drier to employ is

Sipe's Japan oil, most of the other driers being a detriment rather than a help, especially to red-lead paints.*

Best Colors for Paints

Practice seems to have decreed that dark paints are more suitable for bridges than light ones, notwithstanding the well-known facts that they absorb much heat, and that excessive heat is one of the most active agencies in paint deterioration. The main reason for the reluctance to use light paints is their tendency to fade and to show the dirt that inevitably accumulates on all bridge metal; but the fading is avoidable by a proper study of the finishing coat, and it cannot be denied that dirt shows more or less on dark paints as well as on light ones. It must not be forgotten that the selection of paint color for any bridge is a matter of æsthetics as well as of expediency; for in some structures dark colors give the finer effect, and in others light ones provide a better appearance. An old favorite color of the author's is olive green; and he has employed it on a number of occasions with satisfaction to all concerned. Canary yellow and pearl gray are often suitable colors for the finishing coat.

There is one important point about paint colors that should never be ignored, viz., that, no matter how many coats are given to any steelwork, all of them should be of essentially different shades or colors, in order to make sure that all the coats called for are really applied. In one of his early bridges, the author caught the erection superintendent trying to palm off on him a single coat of rather thick paint instead of two field coats of the same color. Ever since then his bridge-erection specifications have called for distinctly different shades or colors of paint coats.

Elasticity of Paint Coats

The matter of elasticity in paint coats is one of extreme importance; because, if neglected, the paint is liable to crack and permit moisture to reach the metal, thus starting rust. As before indicated, there should be a gradual increase in the elasticity of the different coats from the priming one outward.

Covering and Spreading Powers of Paints

One of the most effective claims of paint-selling agents is that the special paint which they handle has a very large spreading power; and this

*Concerning driers Dr. Sabin writes as follows:

Three or four years ago I sent a circular letter, asking confidential opinions as to driers, to about a dozen of the chemists of the really big mixed-paint companies. Without exception they agreed that if price is left out of account the best drier is free from rosin, and contains lead and manganese, from three to twenty-five times as much lead as manganese, but must contain both.

I myself would not object to a drier made by a responsible concern, which contained a little rosin, as most commercial driers do. The trouble is to draw the line.

appeals strongly to most of their buyers—usually erection contractors—who recognize that the greater the spreading power the smaller the quantity of paint required, and, consequently, the less the cost. The bridge owner, on the contrary, is not interested in having his contractor use paint of the greatest possible spreading capacity, because the greater the said capacity the thinner the coating—and the thinner the coating the sooner will it become disintegrated by moisture, gases, etc. Of course, one can go to the other extreme by laying on the coat too thick, in which case it will run on vertical surfaces and will be too slow in drying on horizontal ones. Ordinarily, single coats of dried paint vary in thickness between one five-hundredth and one one-thousandth of an inch; and to produce this the spreading capacity is from eight-hundred to sixteen-hundred square feet per American gallon of paint.* The inspector of painting on bridgework, acting solely in the interests of the owner, should endeavor to have all paint applied just as thickly as it can be used without flowing on vertical surfaces. It should be daubed on thick at first, then gradually worked out by careful brushing so that it will flow into all the pores of the metal.

Distinction should be drawn between covering capacity and spreading capacity of paints. The former refers to the hiding capacity of the coating in relation to the surface on which it is applied, and is measured by certain standard tests on white surfaces, while the spreading capacity refers to the number of square feet of surface that can be covered by an American gallon of the paint.

Cement Paints

Portland cement as a pigment in bridge paints has begun to come into vogue of late years, the principal manufacturers of it being the well-known firm of Toch Brothers, and their product being designated "Tockolith." The author has not yet tried this brand of paint on any of his bridges, hence cannot speak from personal experience concerning its efficacy; but the fact that such a prominent bridge engineer as Dr. Gustav Lindenthal has used it on some of his largest structures is a guarantee that it is a first-class protective agent. However, Dr. Lindenthal is not willing to go so far as to state that Tockolith is superior to red-lead paints of best quality; for the author asked him the direct question and he would not reply affirmatively. Toch Brothers have issued a very interesting little pamphlet "R.I.W. Steel Preservative Paints" (R.I.W. meaning "Remember It's Waterproof); and the reader is referred to that publication for further infor-

* Dr. Sabin prefers to adopt an area of 700 square feet per gallon of paint for red lead coatings, and states that graphite paints are supposed to cover from 350 to 600 square feet per gallon. He claims that red lead makes a stronger and more impervious film than anything else, hence can be safely put on thinner, and that three red lead coats, covering 700 square feet per gallon, make a final film seven one-thousandths of an inch thick.

mation on the subject. If the claims made by its writer are correct, Tockolith has all other paints beaten for bridgework.

Speaking of the claims of paint manufacturers and agents concerning the excellence of their products, reminds the author of an amusing story once told him by Dr. Sabin; and, as it has never yet appeared in print, it is here put on record.

The late Mr. D. D. Carrothers, of the Baltimore and Ohio Railway, used to tell of a well-known paint manufacturer, a most accomplished and adroit salesman, who persuaded him to buy a considerable quantity of bridge-paint, which did not turn out well. When he came around the next year, the engineer told him to get out; his paint was worthless. "I know that," was the unexpected reply; "don't you suppose I know all about my own paint? We tested that paint thoroughly, as I thought; but, when it came to practical use, it developed unforeseen defects, and I have had trouble—the utmost trouble—in finding their cause. But we have found and corrected it, and now I have a paint that is absolutely all right; but I had a dreadful time with it last year." He was so plausible, convincing, and persistent that he went away with another good order; which, however, turned out no better than the first lot. "But," said Mr. Carrothers, "I expect he will sell me some more this year!"

Linseed Oil alone for Shop Coat

A quarter of a century ago it was quite usual to employ boiled linseed oil without any pigment therein for the shop coat on structural steelwork; and, in conformity with the fashion, the author followed the custom when building the Union Loop and the Northwestern Elevated Railroads of Chicago. In so doing he learned by sad experience the fallacy of the practice; for, owing to a sudden shortage of funds, the Company had to close down all work instanter, leaving a mile or more of metalwork erected and unpainted. When construction was resumed a year or two later the steel was in an awful mess, and required an immense amount of labor to clean it. In all probability, that portion of the structure actually lost a portion of its vitality through this unfortunate circumstance. Thereafter, of course, the author refused to allow linseed oil alone to be used for a shop coat; and eventually the custom went out of fashion.

Climatic Influences on Paints

Two decades ago very few persons recognized that climate had anything to do with the durability of bridge paint, and that a brand which was effective in one locality might fail utterly in another, even if the material were taken from the same barrel. During the late nineties the author secured very good results from the Detroit Superior Graphite paint, finding that it could be relied upon for five years before a new coat was needed. All the bridges where he employed it, however, were located in the north

until he built one across the Red River at Alexandria, La. About the same time he was constructing over two hundred bridges on the line of a certain Mexican railway; and, although he experimented there with a number of paints, he used at first the Detroit product on most of the metalwork, for the reason that it had given him such satisfaction previously. It had not been applied in the field on these Mexican bridges more than a year before it began to fail and to go to pieces rapidly; and about the same time the paint on the Alexandria bridge showed rapid deterioration.

Of course, other paints were immediately adopted for the remaining Mexican bridges; and the manufacturers of the Detroit paint were notified of the trouble. In consultation it was decided that the paint furnished by the company, while excellent for dry climates, was unsuited for very damp ones, especially where the dampness was combined with heat. Thereupon the Company instituted an elaborate and extensive series of practical tests, and learned therefrom how to manufacture paints suited to all kinds of climates, thus regaining for their product the high reputation for excellence which it had previously enjoyed. Today it is one of the best finishing coats for bridges that the market affords; but the Company's agents are now very careful to enquire where any paint is going to be used before they sell it.

Of all the paints tested on the Mexican bridges previously referred to, there was only one that proved satisfactory for the damp climate of the *tierra caliente;* and, strange to say, that one was located in the worst possible place for paint endurance, viz., close to the elevation of the salt water of the Gulf of Mexico at the very mouth of a river and not far from the city of Vera Cruz—a place noted for its disagreeable, muggy climate. The paint referred to was Z. P. Leiter's "Air-Drying, Salt-Water-Proof Paint." It had proved to be the best of some twenty standard paints tested at Tampico, Mex., for a large wharf there by the late A. J. Tullock, a noted bridge contractor and the founder of the present Missouri Valley Bridge Company of Leavenworth, Kan. It was because of Mr. Tullock's recommendation that the author tried it at Boca del Rio.

Some six years or more after the bridge was built and finally painted, the author received a letter from the General Manager of the railroad company asking the name of the brand of the paint, and stating that, in spite of its being often drenched with salt spray, it appeared to be in as good condition as new paint.*

Soon after that the author was engaged on the building of an important bridge, having the bottom chords quite close to the salt water, at Vancouver, B. C.; and, naturally, he arranged to use the Leiter paint. The metalwork was being manufactured in the shops of the Dominion Bridge Com-

* It has been stated that, since the death of Mr. Leiter, the formula for his Air-Drying, Salt-Water-Proof Paint has been lost. If such be the case, it is truly regrettable, for it certainly was a most effective protection to steel against the attacks of salt-water.

pany near Montreal, during the winter; and, when the paint was applied to the cold steel, it would not adhere, consequently another brand had to be employed.

Mr. Howard P. Quick, Consulting Engineer, formerly Mechanical Engineer and Designer for the Pearson Engineering Corporation and Associated Companies in Brazil, Mexico, Canada, Spain, and the United States, has very kindly furnished the author with the following information concerning the developing of a suitable paint for steelwork of all kinds to meet the abnormal climatic conditions of Brazil at a place where no paint of any kind imported from either America or Europe could endure the tropical humidity. Various American engineers on Brazilian Constructions, recognizing the gravity of the situation, combined forces, and had the chemists of the Rio Janeiro Gas Company experiment for them in their search for a paint that would meet the conditions. That company possessed an up-to-date, by-products, coke-retort system, and produced therefrom plenty of tar and other residues; consequently its chemists set to work investigating the question on the basis of utilizing these by-products, and finally developed a tough, adhesive, elastic, and glossy-surfaced paint which had all the qualities desired, barring appearance. The composition as reported to Mr. Quick by the American Chemist, Dr. Harrop, General Manager of the *Société Anonyme du Gaz*, was as follows:

1 part of Portland cement.
1 part of kerosene, and
4 parts of gas-works tar.

The cement was first mixed with the kerosene, and then tar was added to develop the consistency required. This produced a glossy, quick-drying paint of a dark, greenish-black color which dried as hard as rock and was unaffected by weather or temperature changes.

Such a paint ought to be quite inexpensive; and it should be tried in *other tropical climates than that of Brazil and also in the Gulf States of the U. S. A. The author surmises that it might* not withstand well the cold winters of our northern states; but it would be well to give it a trial there.

To-day all first-class paint-manufacturers recognize that it is absolutely essential to know about the peculiarities of the climate where any structure is to be erected before starting to manufacture the field coats of paint therefor.

SPRAYING OF PAINT

Spraying paint on bridge metal is a lately-developed custom; and concerning the satisfactoriness of the process there are both pros and cons. It involves a wastage of paint that is unavoidable, but which may be kept down to reasonable limits by constant care and attention; and it is claimed by some painters that it does not work the paint into the pores of the metal

as well as does first-class brushwork. Again, spraying is a messy process, and much paint is likely to go to places for which it was not intended. One great advantage which it has is that it will reach locations difficult of access by the brush, and which, in consequence, are often improperly protected. As far as ultimate total cost is concerned, it is probable that the saving in labor will about offset the wastage of paint.

Cleaning of Metalwork for Shop Coat

Unless metalwork is thoroughly cleaned before the shop coat of paint is applied, the endurance of the protection will be short; hence it is truly economic to ensure that the cleaning is effectively done, so as to remove all dirt, rust, scale, and grease. The time elapsing between cleaning and painting should be made as short as possible; because it does not take long to start fresh rusting on cleaned metal. As to the methods for shop-cleaning, hand-work ought to suffice; for sand-blasting should be unnecessary. If the metal is very badly rusted, it generally establishes sound evidence of carelessness on the part of somebody who ought to be held responsible for its injured condition.

Strictly speaking, all rolled material for bridgework should be taken from the mill to the shops with the least possible delay, and should be stored under proper shelter from the elements, in order to avoid rusting; and it would be in the line of true ultimate economy to give it a coat of linseed oil soon after it comes from the rolls. Most steel manufacturers and users will claim that these precautions are unnecessary, that the coat of oil would be troublesome to put on, and that the storage sheds would cost a lot of money. These objections, of course, are important; but the author is of the opinion that, in the interest of true economy, they will ultimately be overcome, and that sometime in the future all proper precautions will be taken on important steel structures to protect effectively against rust the metal that is to be employed in their manufacture.

The torch should first be used freely in cleaning metal in the shops, for the double purpose of burning any grease that there may be on it and to remove all moisture; after which should follow scraping with steel brushes, file scrapers, and putty knives. Any heavy seed rust which has formed cups in the metal should be chipped out by hammer, care being taken to avoid all unnecessary cutting of the steel. This cleaning should be most carefully watched by a competent and reliable inspector whose compensation comes wholly from the owner; but too often it is done in a careless or perfunctory manner by ignorant foreigners who take no interest in performing their work well. It used to be the custom in some bridge shops to turn over the painting to newly-imported Hungarians, who were the cheapest laborers on the pay-roll; and, as a result, the author has seen metal delivered at site with the shop coat of paint overlying in large areas half an inch of frozen mud. Conditions in this respect today are undoubt-

edly much better than they were twenty-five or thirty years ago, but there is still much room for improvement.

PICKLING

Pickling is an effective way to clean steel; and it is much used in electroplating and enameling, but not for bridgework. The reason is the trouble and expense which it involves. In applying the method, it must be remembered that when metal is immersed in a hot bath of either sulphuric acid or muriatic acid, it will be necessary, immediately after removal therefrom, to neutralize any portion of the liquid which adheres to the steel.

PAINTING NEWLY-ERECTED STEELWORK

As soon as practicable after a span is erected, provided the weather is propitious for painting, the metal should be thoroughly cleansed from any dirt, grease, rust, or other impurity which it has taken on since leaving the shop; and if, for any reason, any serious rusting has started, all traces of it should be removed. Next, all spots abraded either by accident or during cleaning should be touched up with some of the red-lead paint used in the shop (with, perhaps, the addition of a little Japan drier in order to hasten the job); and, after this spotting has dried sufficiently, the first field coat should be applied. This should be allowed ample time to dry before the next coat is put on.

CONCRETE ENCASEMENT

The encasing of bridge metal in concrete is an expedient which has come into vogue of late years. It is employed in most cases for protecting all metal below the level of the deck against locomotive gases, and is not often used above that elevation. The expedient, though, in one sense is quite old; because, for several decades, column feet of trestles and elevated railroads have been protected by enclosure in the concrete of pedestal tops.

In order to make such protection truly effective, the concrete should be water-proof. This result can be accomplished in a number of ways. A water-proofing membrane gives the most certain results; but in many instances it will be best to adopt a rich mixture and to add to the cement some five or ten per cent of its volume of hydrated lime. This matter of water-proofing is treated at length in Chapter XLIII.

As concrete is heavy, its use for a metal protector adds materially to the dead load of structure, thus necessitating the employment of more steel to sustain it; hence it behooves one to make the coating as thin as practicable—and yet not too thin, because very thin concrete may not afford the requisite protection, unless it be placed by pneumatic gun, in which case it is termed gunite.

Gunite

There is an economic question involved in choosing between ordinary concrete and gunite for metal protection, the former being cheaper per cubic unit but requiring a larger volume and, consequently, more metal to sustain its greater weight. The gunite is much more dense than ordinary concrete, and hence affords better protection against moisture. In order to prevent its cracking under changes of temperature, it generally requires coarse wire mesh or expanded metal to hold it together. It adheres so closely to structural steel that, in order to remove it after it has fully set and hardened, chiseling is necessary. Gunite should be at least one inch in thickness. When shot horizontally or vertically upward the covering is strong and uniform; but, when shot vertically downward, sand-pockets are likely to form, unless the operation be carefully watched and all improperly-cemented material instantly removed.

Treatment of Steel that is to be Encased in Concrete or Gunite

In most cases, sufficient attention is not paid to the covering of metal which is to be buried in concrete; because, if it is given a coat or two or ordinary paint, the concrete may adhere to the said paint all right, but the latter may eventually separate from the metal, thus loosening the whole protection and either lessening or destroying its efficiency. In the old days, when the question was much more simple than it is now (involving, as it did, only the burying of anchor bolts or anchor metal in the concrete), the author solved it for his work by giving the metal a coat of boiled linseed oil at the shops, and by scrubbing it off at site just before placement. That method would be impracticable today on account of the large amounts of steel to be protected; hence one might put on the ordinary shop coat of red-lead paint and take it off after erection by means of a sand blast, thus leaving the clean metal exposed to the concrete or gunite.

Toch Brothers, however, claim that their No. 1087A "R.I.W." paint, which contains no saponifiable oil and, therefore, avoids all chemical action between the concrete and the paint, will make a permanent bond between the steel and the concrete. Some other manufacturers make similar claims for special products of their own.

Protection Against Brine Drippings

One of the most destructive agencies in respect to bridge metal is brine drippings from refrigerator cars; and, as yet, no satisfactory protection against it has been found. The metal most injured is that in the top flanges of stringers and cross-girders; but the webs and bottom flanges thereof and the buck braces suffer also more or less. Of course, the ideal method of protection would be to catch the drippings in receptacles on the cars and thus

prevent their ever reaching the bridges; but this has not yet been done. It is a matter which should be investigated promptly; and, if an effective remedy for the trouble be found, it should be utilized without delay by all railroads, because the damage done by the drippings is not confined to the floor-systems, but in deck truss-bridges extends also to the upper chords. The total value of the annual damage to all the steel bridges of North America by brine drippings must amount to a goodly sum.

The railroads are now struggling with the owners of the refrigerator-car lines in an endeavor to force them to lead the brine to one side so far that it will not drip on the track rails and their fastenings; but nothing of any account is being done to protect steel bridges with open timber decks, excepting sometimes to cover each stringer and each floor-beam with a wide plank or to daub thickly the top surface of the endangered metal with some paint or other compound that has no chemical affinity for the brine.

Protection Against Locomotive Gases

Another deadly agent of destruction to steel bridges is the gas from steam locomotives, especially when they pass beneath the structure, in which case the gas collects in semi-enclosed spaces where it remains and steadily attacks first the paint and then the steel beneath.

As before mentioned, the surest method of protection against such attack is to enclose the entire metalwork below the elevation of track-rails in concrete or gunite. Another method is to use special paint beneath the floor and to hang wooden or sheet-iron protecting platforms close to the deck. If the tops of the smoke-stacks of the locomotives come very near to the floor of the bridge above, especially where there is a heavy up-grade, cinders are driven out of the stacks at high velocity, combined with smoke and steam, thus forming the most effective possible kind of blast for cutting first the protection and then the steelwork itself.

Causes of Paint Deterioration

The main causes for deterioration of bridge paint, as stated by the late Mr. Houston Lowe, are as follows:

1. Water. (Dissolution.)
2. Action of light and heat. (Chemical and physical change.)
3. Chemical action between pigment and binder. (Disintegration.)
4. Abrasion or mechanical injury. (Motion.)
5. Action of deleterious gases. (Foul air.)

It is impracticable to keep water away from a bridge, but the design can be so made that there are no pockets to hold it, in which case it quickly runs off and evaporates.

The deleterious action of light and heat can best be combated by a proper choice of color for the finishing coat.

Disintegration by reason of chemical action between pigment and binder is a matter for the paint-manufacturers' chemists to take care of; and this has already been done by all of the first-class companies with more or less success.

Abrasion of bridge paint, except by the before-mentioned cinder blast from steam locomotives, is something which should seldom occur, because nothing should be allowed to strike the metalwork of a bridge hard enough to disturb the paint thereon.

In respect to the action of deleterious gases, that question has already been discussed at some length herein.

How to Care for Incipient Failure of Paint

As all bridges should be submitted to careful inspection at short intervals of time, any incipient failure of paint should be quickly discovered. If the failure pertains to the field coats only, it will suffice to cover the defective parts with a layer or two of the final coating; but, if it extends into the shop coat, the spot affected should be covered with red-lead paint of best quality, and afterwards, for the sake of appearance, re-covered with some of the finishing-coat paint. If the metal is exposed and rusted, which will not occur if the structure is properly examined at intervals of time not too great, some scraping will be necessary before the red-lead paint is applied.

By careful attention of this kind the life of the paint can readily be extended from twenty-five to fifty per cent, as compared with what it would be without such attention.

Determination of Time for Repainting

When the partial failures begin to come so rapidly that the retouching process is expensive and troublesome, and especially when the attacks appear to be liable to reach the metal through the shop coat, it is about time to give the structure a thorough cleaning and one or (better) two coats of paint. The psychological time for doing this can only be determined by an experienced bridge man—preferably the Superintendent of Structures of the railroad, state, county, or municipality.

Cleaning of Metalwork Preparatory to Applying New Field Coats

The cleaning of the metalwork, if it has been properly cared for, will not prove to be a serious business; but, otherwise, it will involve a rather drastic operation. If possible, the shop coat of red lead should not be disturbed, but should be effectively re-covered. In places the torch may have to be used; but, as it cuts to the quick, its use should be avoided whenever practicable. Similarly, the sand-blast should not be adopted,

unless actual rusting has started, for it eats into the metal; and some good authorities claim that where it is employed the paint will not adhere as long as it will where hand cleaning alone has been adopted. The use of the sand blast deposits a lot of sand in numerous corners and pockets of the metalwork; and it should all be carefully brushed therefrom before any painting is done.

Application of Paint after Cleaning

The following suggestions concerning field painting, if followed, should lead to economic results in maintenance of bridges:

First. Avoid both cleaning and painting in wet or very cold weather. A few months' delay will seldom do any real harm to the metal or even to the priming coat, unless, perchance, the bridge has been neglected to the extent of actually permitting rusting to start.

Second. Never do much cleaning ahead of the painting, because a spell of bad weather may come on and last so long that such cleaning will have to be repeated. It is not only the extra expense of doing the cleaning twice which is uneconomic, but also the possible injury to the exposed metal from rusting and pitting.

Third. Provide large and safe platforms for both cleaners and painters. It is true that these may be expensive in both first-cost and handling, but their use will enable the workmen to do a much greater amount of work per diem—and better work—than if they were not effectively, safely, and comfortably supported. Again, there is to be considered the reduced danger to the lives of the workmen; and as a killed employee generally costs the company $5,000, and an injured one whatever amount he can persuade the company to give him or the court to award, it is certainly economical as well as humanitarian to reduce the danger to a minimum.

Fourth. Give the first field coat a chance to dry thoroughly before applying the next one. In a long structure or a large one, requiring weeks to clean and paint, this restriction will work no economic hardship; but in a short bridge it will, often necessitating the moving of the painting gang to another structure and returning later to apply the second coat.

Fifth. When the amount of cleaning and painting is large, it will be economic to divide the gang permanently into groups of cleaners and painters; but when the bridges are small, or when the total amount of work is not large, all the workmen should be trained so as to become proficient in both of these kinds of labor.

Factors that Affect Results in Painting

As stated by Mr. Houston Lowe, the principal factors that affect results in painting are as follows:

1. Location of the structure, for example, seaboard or inland.
2. Kind and condition of the surface.

3. Quality of the paint and its temperature.
4. Workmanship of the painter.
5. Number of coats applied and their sequence.
6. Time allowed to elapse between coats.
7. Atmospheric conditions when painting is done.

By giving all these factors due consideration when handling a job of bridge painting, and by striving in every way to accommodate the work in the best manner possible to the governing conditions, one can often effect a decided economy.

Economic Observations Concerning Painting in General

The price of paint is a matter which seldom needs much consideration from the economic standpoint, unless when debating on the choice of two or three kinds of nearly equal quality; because painting is so much more expensive than the paint itself that a very little extra life of the coatings will out-weigh in economic importance a large difference in the cost of the material. Generally, one should determine what kinds of paint would probably be the best for any job, then pay for them whatever is necessary, unless it should occur that the seller is extortionate in his demands, which is not likely to be the case when dealing with first-class business men.

It does not pay to dicker about the price of paint, because the manufacturer is accustomed to "cutting his coat according to his cloth." The author remembers a glaring case of this reprehensible trick that occurred in the nineties. He had been using very satisfactorily a certain magnetic-iron-oxide paint that was not expensive, and recommended it to a contractor-friend of his for an elevated railroad line. Some three years later the paint agent dropped into the author's office and asked him to specify the brand in question for a large piece of work then about to be let. The request was refused—much to the surprise of the agent, who exclaimed "is not our paint that you used on the Blank City train-shed giving good service after four years of use?" The reply was: "Yes; but how about the condition of your paint on the elevated railroad in the same locality—is it not already going to pieces with only two years' service?" In answer to this question the agent had to explain that the Contractor had jewed down the price and, therefore, the paint had to be adulterated in order to meet the cut. Thereupon the author refused to have any more dealings with such a dishonest manufacturing company, and never again adopted the said paint on any of his metalwork.

A layer of good paint is about three times as thick as a layer of the linseed oil in which it is mixed, and the increase in thickness of the former is in direct proportion to the fineness of the pigment; hence it is economic to have the said pigment ground as fine as possible, for it is a fact that, with the same weight of oil and the same weight of pigment, the greater the

volume of the latter—i.e., the finer it is ground—the more slowly will the paint dry and the longer will it endure.

The correct and economic theory of metal painting is that the paint used for the priming coat should be of a preservative nature, i.e., of such a character that it will not only possess the power of inhibiting the corrosion of the metal but also that of absolutely excluding air and moisture therefrom, and that the other coats should be of a protective nature, i.e., that they will protect the priming coat from the deleterious action of rain, sunshine, and all other deteriorating agents.

In respect to the character of the pigments for the finishing coats, it is important that they be both chemically and physically inert, that they be ground very fine, and that they have an affinity for linseed oil. It is a matter of minor importance whether the pigment be graphite, lampblack, charcoal, oxide of iron, or what not, as far as durability is concerned. Where only three coats of paint are to be used, it is often advantageous to employ for the middle one a direct mixture of the priming and the finishing paints.

The character of the brush used is an important element in painting; for it is possible to ruin a coat of good paint by applying it with a broad, thin, flat brush. Instead, the paint should be well rubbed into the surface with a stout, full, round, bristle brush. "Less paint and more painting" should be the slogan for every painter who desires to do good work.

A temperature of about 70° F., combined with an atmosphere free from moisture, makes the ideal condition for both the applying and the drying of paint. The matter of humidity is of even greater importance than that of temperature; because nothing retards drying more than dampness and darkness.

For places that are badly ventilated, where sunshine does not reach, and that are damp or at times filled with steam or locomotive gases, it is economic to use a special kind of coating, preferably a varnish or resin paint so composed that it will dry rapidly.

This dissertation on paint and painting could be carried on almost indefinitely; but, if it were, the author might properly be open to criticism for departing from his subject of economics. It is possible that, even as it is, some of the readers of this chapter will accuse him of that fault. If so, he would reply that anything which treats of how to prolong the life of bridge paint or to protect the metal effectively against corrosion is directly in the line of true economics.

Summarizing in a very few words all that has been said herein upon the subject of "Economics of Metal Protection," it may be stated that it is always truly economic to use the best protective agencies procurable, irrespective of their cost, and to spend without hesitation all the money requisite for taking the very best of care of the metalwork in all first-class railway and highway bridges—or, for that matter, in any other bridges that are worth saving from speedy destruction.

CHAPTER XLIII

Economics of Water-proofing

The economics of water-proofing solid-floor bridges might properly be considered in the chapter on "Economics of Maintenance and Repairs," or in that of "Economics of Metal Protection"; but the subject is one of such importance that it truly merits special treatment in an independent chapter.

Very few engineers are sufficiently conversant with the water-proofing of bridges to permit of their writing authoritatively upon the subject of its economics, and the author's experience therein certainly has not been very extensive; hence he deems himself exceedingly fortunate in having secured the aid of Mr. J. B. W. Gardiner, President of Gardiner and Lewis, Inc., of New York City, who has made a special study of the subject, extending over several years, and who has very kindly prepared for him the substance of the following dissertation:

The economics of water-proofing means this: Is or is not water-proofing worth while financially? does the effect produced or the protection afforded the structure by water-proofing the floor justify the expenditure involved? It must be assumed at the outset that the same care has been taken with the water-proofing as is taken ordinarily with the steel or other structural material, i.e., that the bridge was designed to be water-proofed, that a careful selection of materials was made, and that those materials were installed by men skilled in that particular trade, or at least were placed under careful supervision. These conditions are no more stringent or unreasonable than those which apply to the design, fabrication, and placing of other materials that go into the structure. The initiatory presumption is, therefore, entirely proper.

Whether water-proofing as applied to bridge floors is a profitable investment depends on several factors. These are:

a. The proportionate original cost of the water-proofed bridge as compared with the total cost of the bridge without water-proofing.

b. The probable life of the water-proofing and the probable cost of its renewal.

c. Its effect on the life of the structure itself and on the cost of maintenance and repairs.

The original cost of water-proofing is a function of a number of different factors. The cost of the material itself delivered at site, that of the direct labor involved in placing it, the cost of such provisions as are made for flashing along the lines of termination of the water-proofing, the cost of the protective or armor coat, the additional depth of flooring and the greater strength that must be provided in steel bridges to carry the dead load of the water-proofing and its protection, and the accessibility of the site of the work, are all considerations that influence the solution of this question of economics. It is readily seen that the cost of water-proofing must be a fundamentally variable factor, as well as one which fluctuates with the building-material market. Moreover, the cost per square foot is usually greater in the case of steel bridges than when the protection is applied to concrete viaducts; because, in the latter, the increase in the dead load does not materially affect the design, and also because in concrete bridges the protection coat is usually a part of the paving-floor system (except in bridges of the solid-spandrel, earth-filled type), and hence involves no additional cost. At present-day prices, however, a cost of 40 cents per square foot for water-proofing steel bridges or 35 cents per square foot for water-proofing concrete bridges is a fair average.

The relative cost, i.e., the cost of the water-proofing as compared with the total cost of the structure, is also variable. In the case of unique steel bridges, such as the Hell Gate Arch or the Quebec Cantilever Bridge over the St. Lawrence River, or in unusual concrete bridges, such as the Tunkhannock Creek Viaduct on the D. L. & W. R. R., this percentage cost is abnormally low, because of the extremely high cost of those structures per square foot of floor area. But such structures as those are so unusual that we may ignore them in our cost considerations, and base our results on the more common types of bridges.

Data gathered from several eastern railroads show that, of the total cost of a steel bridge, four per cent is properly chargeable to water-proofing. In computing this figure various standard types of bridges were considered i.e., concrete-filled trough-floors, plate-girders with transverse I-beam floors, longitudinal I-beam floors with steel-plate, and deck-girder bridges. Cost data were also gathered for different periods—before, during, and after the war—and the results were averaged. It might be remarked in passing that the advance in water-proofing costs has not kept pace with the advance in other elements of construction. For example, in 1914 the steel in a certain bridge cost $73 per ton erected and the water-proofing 28 cents per square foot. In 1917-1918 the steel in a bridge of exactly the same type cost $143 per ton erected, while the water-proofing cost but 35½ cents per square foot. Thus, although the steel increased in cost ninety-six per cent, the water-proofing increased only twenty-seven per cent. When it is stated, therefore, that the cost of water-proofing averages four per cent of the cost of the structure, we are taking a figure which is really extreme and which we may safely assume will not be exceeded, except in isolated and very

extreme cases. To look at this percentage in another light, the interest on the cost of a bridge for one year at four per cent will pay for the water-proofing.

In concrete bridges the relation between water-proofing costs and total costs is apparently a variable with wide limits, because the character of the pier-work exercises such a great influence on the latter figure. As a matter of fact, however, for certain types of bridges this relation is surprisingly uniform; but distinction must be made between two classes of concrete bridges, in order to arrive at the proper ratio. This is because of the cost of the protection or armor coat placed on the water-proofing. Such protection is always adopted for concrete railroad bridges of whatever type, and for earth-filled, solid-spandrel highway-bridges. These two then may be considered together. On flat-slab highway bridges, the protection coat is frequently omitted; and, if used, it is little more than a grouting course under the paving, which might be required even if the bridge were not water-proofed. This type of bridge constitutes the second class.

In the first class the cost of water-proofing bears about the same relation to the total cost as was found in the case of steel bridges—i.e., an average of four per cent. Isolated cases were encountered in railroad viaducts where the cost of water-proofing ran up to eight per cent; but this was exceptional, the usual cost being well under five per cent.

On flat-slab highway-bridges remarkable uniformity in relative costs was found. Ninety per cent of the cost figures which were analyzed showed the expense of water-proofing to be between one and a half and two per cent, and only a single bridge showed a greater cost. This last figure, therefore, may safely be taken as a fair average. Thus in all bridges, steel or concrete, except flat-slab-floor highway-bridges, the water-proofing cost is four per cent of the total, while in flat-slab-floor structures it is two per cent thereof.

The only element of cost which has not been considered is that of the interest on the investment; but this will not change the ratios which have already been arrived at, since the rate of increase in the cost of water-proofing through the addition of interest charges is, of course, identical with the rate of increase in the value of what the water-proofing is to protect, hence this element of cost may be ignored.

Probable Life of Water-proofing

By the probable life of water-proofing is meant, of course, its effective life,—the length of time it will continue to exercise its water-excluding function. If a bridge has been properly designed with respect to water-proofing details, and if the water-proofing has been properly placed it will fail, if at all, from one or both of two causes,—a break in the continuity of the water-proofing envelope by rupture or otherwise, and the deterioration of the water-proofing material itself by oxidation or rotting. The shock of

impact when a locomotive first passes on a bridge, the grinding wrench when brakes are applied while it is on the structure, the vibration of any steel construction caused by the moving load, the deflection and reverse bending on continuous-span steel-bridges, the changes in volume incident to temperature variations—all these place very severe but indefinite stresses on the water-proofing blanket. At the same time, an engineer, knowing these conditions, can select a material, the physical properties of which will generally meet them. Materials which are brittle at low temperatures should receive scant consideration, because of the probability of their being fractured by vibration in cold weather. Materials which are very soft at high temperatures should likewise be regarded with suspicion, because they cannot be held on vertical or steeply-inclined surfaces. Finally, a material should be selected all parts of which are flexible and elastic, so that they will yield rather than break under the conditions mentioned. To put it briefly, the water-proofing, in so far as physical requirements are concerned, should be selected on the basis of its plasticity or flexibility at all temperatures and its small factor of susceptibility to temperature changes. As to the deterioration of water-proofing, that is almost entirely a chemical matter. The exclusion of all materials that are affected by water, whether or not the water carries acids or alkalies, such as the ligno-cellulose compounds (jute or burlap) and the felts in which tapioca is the binding medium (asbestos felt), the non-employment of those that are physically unstable, such as most of the artificially compounded asphalts and asphalts containing organic matter, and the selection of a material which has to a large degree been pre-aged or pre-oxidized will assure long life for the protection. Indeed, water-proofing materials (asphaltic) placed in 2500 B.C. have lately been found to be still in good serviceable condition. A discussion of the various materials sold for bridge water-proofing is a complicated one and very technical. It is relevant to the subject of this chapter only in so far as it may point out the answer to the questions that have been raised. It is sufficient to say, however, that if a selection of materials is made with regard to the known conditions which must be met, rather than on the basis of initial cost, a life of at least 25 years may reasonably be anticipated, with the probability that this figure will be greatly increased.

It is obvious that, if water-proofing a bridge floor is economically wise, the resultant value of the protection afforded to the structure thereby must exceed the cost thereof. That the said value does exceed the cost may be a difficult proposition to establish by definite figures. As far as is known, no data exist concerning the amount of the damage caused by lack of water-proofing. All that can be done with this question is to indicate the agents which attack bridge floors and the effect of water-proofing in warding off such attacks, leaving it to the judgment of the individual to decide whether the probabilities in the case justify the expense which water-proofing involves.

For convenience in treatment, the effect of water on steel alone will be

considered in connection with steel bridges, the effect thereof on the concrete floor of such structures being discussed in connection with concrete viaducts. The primary function of water-proofing on steel bridges is to furnish protection to those steel members which, because of the very existence of the solid floor, are not accessible for ordinary maintenance. They cannot be painted or otherwise guarded from moisture—as they are, so must they remain. On railroad structures the flow of water to the drains is seriously interfered with by the ballast, which, to some extent, acts as a dam to hold the water on the surface of the concrete floor. The said concrete floor may thus become saturated, because of the moisture being held for a considerable time against the steel—an ideal condition for destructive corrosion.

Brine-drip from refrigerator cars is a singularly active corrosive which is a cause of trouble and annoyance to bridge engineers everywhere, its effect being very frequently seen on the top flanges of stringers and floor beams, where it is extremely destructive. Not only are these affected, but also in half-through, plate-girder bridges those portions of the webs of the girders which are covered by the concrete are also subject to corrosion, as the joints where the concrete meets the steel invariably open sufficiently to allow water to enter.

It is unnecessary, though, to dilate further on the action of either atmospheric water or brine-drip on steel. The facts are well known and universally recognized, and it is conceded that many bridges have been seriously damaged by rust, even to the extent of having to be replaced. This deterioration can be prevented on the exposed metal by painting; but the only protection that can be given to those members which are not exposed is water-proofing. The actual money damage resulting from failure to water-proof cannot, for lack of dependable data, be definitely stated. Certain it is, however, that, if a steel bridge is left unpainted long enough, the sectional areas of the metal will be so reduced that the structure will become unfit for use and ready for the scrap heap. And if this is true of the exposed members from which the water dries out quickly, how much more true is it of those members which are subjected to a greatly aggravated condition! At just what period inaccessible and unprotected steel will become unsafe no one can say; but the fact that so many and such important members cannot be inspected so as to determine their condition would appear to impose upon an engineer, from the standpoints of both public safety and economy to his client, the duty of protecting the metal by efficient water-proofing. With increasing live loads, it is all the more necessary to maintain the full strength of the steelwork, and not to allow of its weakening by preventable deterioration.

From the foregoing it is evident that water-proofing is a necessity for bridges which contain structural steel embedded in the concrete of the floor, and for those having members encased in concrete or gunite. Failure to follow this practise has caused loss in some instances, the encased

members being seriously weakened, and the encasement being split off by rust. It is not so important when the concrete slab merely rests on the tops of the stringers without encasing them; for the destructive effects are then limited almost entirely to the slab itself and to the tops of the stringer flanges.

The question as to whether reinforced-concrete viaducts should be water-proofed is somewhat more complicated than that for steel bridges, owing to the fact that there is not the same concurrence of opinion as to the injurious effect of water on the structure. It may be stated, though, that the water-proofing of concrete viaducts has lately become a common practice with the railroads of this country and with many of the private consulting engineers, as well as with a number of state, county, and municipal officials. Certainly, those engineers who include water-proofing in their specifications are actuated by the same desire to produce creditable work, the same loyalty to the interest of their clients, as those who do not. This being the case, on what theory and for what reasons do they consider it justifiable to make this addition to the cost of the structure? Just *why* should a reinforced-concrete viaduct be water-proofed?

That there is virtue in water-proofing a flat slab, even though well pitched for drainage, is shown by the experience of the country with concrete roads. It cannot be denied that such roads, well blanketed with asphalt, give longer and better service, with less cracking and other evidence of disintegration, than those not so blanketed. While this is partly due to the fact that mechanical wear is eliminated, the protection of the concrete from the action of frost and freezing water is an important factor. The drainage problem on a road is much more simple than that on a bridge floor. The standard width of road is generally but 18′, so that the area to be drained is only 9′ wide—the distance from the crown to the curb. The areas on a bridge floor are generally much larger; and since the water must be conducted to small down spouts, instead of to an open trench, as is the case with a road, the problem in bridgework exists in a much more aggravated form. If, therefore, it is found that water-proofing a concrete road by means of an impervious blanket of asphalt protects the road, prevents its disintegration, and prolongs its life, how much more urgent is the need for water-proofing a bridge floor?

A brief analysis of the more important of the disintegrating effects of water penetrations on concrete will serve to bring out the reasons why from an economic standpoint—i.e., from the standpoint of preservation and consequent reduction in both annual-replacement reserves and maintenance charges,—the water-proofing of concrete bridges is profitable.

Water is a universal solvent, affecting, of course, some materials more than others. There is in concrete some soluble matter; and, if water be permitted to pass completely through, such soluble matter is gradually removed by a leaching process. This is shown by the fact that water, after it has passed through concrete, will invariably give alkaline reaction,

indicating that it has carried away some of the lime or other alkali content of the cement. Coupled with this is an erosive action which, while slight, is nevertheless present. Particularly are both solvent and erosive actions aggressive at the construction joints, as is evidenced by the excrescence of magnesia and other salts on the under surface at these places. This is because at the joints there is almost always found a film or deposit of laitance, which is loose in texture, non-coherent, chalky, and very porous. Water, if allowed to pass through the concrete at the said joints, will both leach and erode the joint walls very rapidly, thus exposing the reinforcing steel to the air with consequent corrosion, and opening the way in temperate and cold climates to the disruptive effect of frost. The result of these combined actions on concrete is certainly the weakening of the structure, as well as the making ready for further and more drastic effects of water disintegration.

In climates where freezing temperatures prevail in winter, there are peculiarly forceful reasons for water-proofing bridge floors. The disruptive force of freezing water is one of the most destructive agencies operating against masonry of whatever nature, whether its mass be mainly natural or artificial. It is particularly harmful where the masonry contains cracks, however small or shallow they may be, into which water can penetrate freely. This again has a most direct bearing on concrete bridge floors. In common with most flat-slab constructions, whether the slab rest on arches or beams, bridge floors are almost certain to develop cracks. They may be merely shrinkage cracks incident to the setting of the concrete, and hence more or less superficial, or they may be expansion cracks extending through the full depth of the slab; but the ultimate results will be practically the same. If only surface cracks exist, these will fill with water, which, on freezing, will break down the surrounding walls, causing the concrete to spall. It will thus be weakened along the line of the crack, which must inevitably be deepened with each repetition of the process, until the injury has extended through the entire thickness of the slab. Furthermore, since the concrete has been weakened along this line, any unusual stresses incident to expansion, or other force, are apt to split the slab at the weakened section.

Where the crack extends completely through the slab, the action is still more serious. In such a case, the same frost action may force the walls of the crack apart a material distance, exposing the reinforcing metal to uninrupted and unavoidable corrosion, thus weakening the entire structure. If electricity be present, the condition will be aggravated by electrolysis, which will not only accelerate the corrosion of the steel but also will soften the concrete, providing it be moist or wet. (See Tech. Paper, No. 19, U. S. Bureau of Standards.) If electrolytic action does occur, the concrete is certain to be split away from the reinforcing material by the mechanical pressure of the forming rust scale, which pressure has been recorded as high as 4,700 pounds to the square inch.

Efficient, well-planned water-proofing is the only sure preventive of these evils.

As provocative of cracks in concrete, the effect of moisture-changes merits attention. It has been fully demonstrated by some most-carefully-conducted tests that concrete expands in volume on becoming wet. This fact may well produce cross stresses in the slab, which will result in the cracking of the surface. If, for example, we have a floor slab 10 inches thick, and if sufficient water falls upon it to wet it to a depth of two inches, the upper two inches will have a tendency to swell or expand, while the lower 8 inches will remain fixed. Along the planes separating these two, therefore, there will be produced a stress which no amount of provision could guard against. Should this cross stress produce surface fissures or cracks, the way is opened for the ultimate disintegration which has been mentioned.

On this point, Hool & Johnson, in their "Concrete Engineers' Hand Book" made the following statement:

> "The expansion and contraction of mortars and concretes, subjected to variations of temperature and moisture conditions, are responsible for practically all failures of these materials under conditions of exposure to the weather. Either temperature effects or moisture effects may be alone operative, or both effects may be combined. . . . In the average situation the introduction of dangerous stresses caused by a tendency to expand or contract is more apt to be due to moisture changes than to temperature changes, because the volumetric variations in the latter cases are less marked."

There is still another factor entering into the subject, to which engineers generally are not inclined to pay sufficient consideration, largely because it is one which is not directly or immediately measurable in dollars and cents, viz., the matter of appearance.

There is no concrete structure which is designed for a greater degree of permanency than a reinforced-concrete viaduct. It is almost invariably an important, indeed a vital, link in a railway or a national or state highway, the line of which does not change in a generation; and it is usually designed to carry many times the load which present conditions render requisite, thus making improbable the necessity for renewal by reason of changes in transportation methods. It stands, therefore, through many years a monument to the man who designed it, as well as an indication of the progressive spirit of the community; consequently, both from the standpoint of the designing engineer and from that of the community, every reasonable precaution should be taken to preserve the appearance of what is naturally a beautiful structure.

If water is permitted to flow freely through a bridge floor, the result will invariably be the excrescence of magnesia and other salts, which appear on the surface in the form of a white "bloom." This is particularly in evidence at the construction joints, and notably at the joints between successive arch ribs. Nothing is more disfiguring to a concrete bridge, nothing is more indicative of careless or incomplete work, than the discoloration of

a structure due to the action of water. It is readily guarded against at small cost; hence, for æsthetic as well as preservative reasons, water-proofing is well worth while.

This consideration, apparently purely æsthetic, has, paradoxical as it may seem, a distinct place in a discussion of the economics of bridge water-proofing. Cities and towns are in constant competition with each other for new industries. Large sums of money are spent annually by these municipalities or their Boards of Trade in advertising their advantages as *loci* for manufacturing enterprises. Visible evidences of a spirit of progress in the community, obvious care in attention to details of appearances of public structures, are impressive and cannot fail to attract the notice of prospective residents. This makes for the wealth and progress of all concerned. When a visitor's first impression of a city is produced by seeing what should be a beautiful monumental construction covered with the disfiguring surface-blemishes which water-penetration produces, he is apt to turn to a competitor whose neglect of detail is not so marked; for to the layman—this term being used to differentiate from the engineer—such blemishes mean more than mere surface disfiguration, because they convey the impression of general disintegration and eventual failure, and reflect unfavorably upon the community or the owner of the structure.

It is not contended that the neglect to water-proof a concrete bridge will always result in its ultimate destruction. Many bridges which are not water-proofed are still intact; few have completely failed. But water-proofing certainly does provide a measure of protection. It is, in fact, a form of insurance. The cost is small—not more than two per cent of the total—so that if it extends the life of a bridge only a year or two, it is worth while, since the prolonged use of the structure is almost always of greater value than the extra cost plus compound interest thereon. When, however, the protection afforded is not merely for a year or two, but for a great many years, the life of the water-proofing being the only limit, and even that being capable of extension by renewals, it is obvious that the additional cost incident thereto is a wise investment.

Although, of late years, the author has been specifying the addition of a small percentage of hydrated lime to the cement used for making concrete in bridgework, with the dual purpose in view of increasing fluidity and reducing porosity, he has not called for blanket water-proofing and flashing, excepting where there was special reason for preventing drip; but, because of the convincing character of the preceding dissertation, he has decided that in future, if his clients can be persuaded to stand for the additional expense, he will adopt every available effective means in order to water-proof his structures thoroughly.

Since the preceding was written, the author's attention has been called to an important paper delivered to the Brooklyn Engineers' Club in May,

1916, by Mr. Albert H. Rhett, C. E., entitled "The Water-Proofing of Structures Subject to Stress from Moving Loads and Temperature Variations." In it Mr. Rhett gives a short, chronological record of the various unsuccessful endeavors to produce an effective water-proofing for bridge floors, leading up to a successful one of his own. In concluding his interesting and valuable memoir he makes the following statement:

> To recapitulate, then: The theory evolved, and which it was attempted to prove, is that, if a structure subject to moving load and temperature variation is to be water-proofed, it can be effected only through the medium of a membrane, incorporated in the floor, which fulfills these two conditions:
>
> (1) The compound element of this membrane must, of necessity, be the eventual factor upon which reliance is to be placed.
>
> (2) The compound, to fulfill its true function, must remain water-proof, non-hardening, elastic, coherent, and adherent at low temperatures as well as high.

CHAPTER XLIV

ECONOMICS OF MILITARY BRIDGING

FOREWORD

By Major General LANSING H. BEACH, Chief of Engineers, U. S. Army.

THE work of our Engineers in France during the World War was something of which the whole American people have good reason to be proud. In this work the members of the engineering profession showed their great versatility and their ability to attack successfully problems previously unknown to those without military training.

There were many things to be learned and some to be unlearned. The Civil Engineer, in general, was forced to revise much that had been inculcated in him during his earliest studies in engineering, and confirmed during the course of his professional practice. Possibly the most important new lessons were that, in a military construction, time is of more importance than any other feature involved; that a structure, especially a bridge, does not have to be of uniform strength throughout, but that, like a chain, it will answer its purpose if its weakest link is just strong enough to stand the strain which will be put upon it; that architectural beauty for its own sake has no place in field operations; and that permanence is a consideration of such slight importance that frequently it does not enter the calculations at all.

The appearance and the lack of strength occasionally gave the new officers of Engineers a shock, inducing a feeling that much of their study and practice had been in vain and that much of their previous experience could help them but little in the construction of emergency bridges. It is, however, the Engineer best trained in civil practice who can build the best emergency structure, if he will but properly evaluate the conditions imposed by military exigencies.

The Engineers in the War were adaptable, but we are all the results of our education and training; and it is, therefore, not amiss to bring to the attention of the members of the profession at large certain principles which they may learn in time of peace and be prepared to apply in time of war, and thus avoid having to acquire this knowledge in the face of the enemy, when the lives of thousands of their fellow citizen-soldiers are at stake. Victory and defeat, the rise and the fall of nations, have often depended upon seeming trifles. It is conceivable that an engineering failure in war may involve the extinction of a state.

The eminent author of this book has made a profound study of the economics that apply to many of the problems which the Civil Engineer is called upon to solve, and it will be widely read by all engineers who are determined to be in the forefront of their profession. It, therefore, seems most fitting that in this book there should appear a chapter dealing with the principles which must apply to a phase of the work of the Military Engineer, work which any one of the many readers may be called upon at some time to carry on.

Colonel P. S. Bond, Corps of Engineers, U. S. Army, was selected to prepare this chapter because of his intimate knowledge of the subject, acquired by study and by practice. He has presented it clearly and logically, pointing out the essential differences which must prevail between the principles that govern the building of bridges, according as they are intended for military use or to serve peaceful purposes.

He has stressed particularly the all important time element in war enterprises, and has shown that what in civil practice might be wilful waste, in war may be the greatest of all economic measures. In fact, so tremendous is the cost of conducting a modern war, it is hardly too much to say that any expenditure of money or of material which will shorten its duration is easily justified. A careful reading of this chapter is, therefore, earnestly recommended to all of those to whom the Army must look for help when next we are called upon to take up arms in the defense of our rights and the rights of humanity.

Economics of Military Bridging

Streams constitute one of the greatest obstacles to military operations, and bridge building is, accordingly, one of the chief duties of the military engineer.

Fundamental Economics of Military Engineering

Military bridge engineering is an adaptation, in a simple and frequently crude and makeshift form, of civil practice to military needs. The fundamental difference between civil and military practice is in their economic aspects. The technical details of military bridges are characterized by extreme simplicity, which is demanded by the conditions under which they must be built. They will present little difficulty to the engineer having a good general knowledge of civil practice.

But the successful practice of military engineering, including bridging, demands a knowledge of the economic principles which are specially applicable to warfare. Mistakes in economic judgment will have more far-reaching and disastrous consequences in war than in peace—there will be greater opportunities for tremendous profits or ruinous losses. A practical

knowledge of economic principles is, accordingly, of greater importance than a knowledge of technical details. It will, therefore, be of interest to consider these principles in their application to warfare.

In war the highest economy is victory, and the greatest waste is defeat; consequently, anything which contributes to victory and evades defeat is justifiable, however great its cost or the incidental waste involved—of course within the limits of reason and common sense. A free and rapid expenditure of available resources in war is not waste, but the highest form of economy when it contributes to early victory. The march of events is rapid in modern war. A few days, even a few hours, have decided the issue of battle. The Commander does not ask his Engineer, "How much will your bridge cost?" but, "How soon will it be ready?"

In civil bridge construction the essential requirements, in their *usual* order of importance, are. initial cost, safety, durability or permanence, time required for construction, and æsthetics. The time required for construction is of importance chiefly in so far as it affects the financial returns on the investment. A considerable time spent in design and other preliminaries to construction, and on the work itself, will usually be amply justified by a material saving in cost.

The Time Factor Substituted for the Cost Factor

In military construction some of these desiderata entirely disappear, and the order of importance of others is reversed. In particular, the time of construction becomes of paramount importance—the time factor is substituted for the cost factor as the principal consideration. In sharp contrast to civil practice, we find that any cost will be justified, if it results in saving of valuable time at a critical juncture. As a consequence of this, the construction of military bridges is conducted with feverish rapidity. The highest achievement of the military bridge builder is a structure just sufficient for its immediate purpose, erected in the minimum time, without undue regard for cost, appearance, or durability. Time is always the chief —often the only—consideration; delay is always inadmissible; and success is the only criterion by which the engineer will be judged.

In civil construction time is of importance; but it is seldom necessary to sacrifice cost, safety, and all other considerations to gain time. A delay for a moderate period will usually not be hurtful. In military operations, on the contrary, a delay of a few days, or even a few hours, may mean failure instead of success. A similar situation sometimes arises in civil bridge practice, as, for example, when it is necessary to restore traffic on an important main-line railroad after a bridge has been destroyed by fire or flood. In such a situation the economics are similar to those of warfare; and military methods of procedure would there be appropriate.

The Principle of "Bare Necessities Only"

Military bridges are always required in the least possible time, and there will very often, perhaps usually, be a dearth of building material.

It is, accordingly, a fundamental economic principle of military construction that *bare necessities* alone should be provided for. An engineer who wastes valuable time and material constructing a bridge wide enough and strong enough to carry heavy motor trucks, when the need of the instant is a simple foot-bridge over which a body of infantry may pass *at once* to a critical point of the field, has manifestly failed to grasp this fundamental economic principle.

Safety and Permanence

The civil engineer "builds for posterity." Whatever the type of bridge adopted, it is usually constructed in as enduring a fashion as the funds available and the material employed will permit. The useful life of the bridge is ordinarily a measure of the skill of its builder.

The military engineer builds to meet the exigency of the moment. He is lacking in skill if he expends time in order that his bridge may endure unduly long beyond the period it is needed, which in no case exceeds the duration of the war, and is often limited to that of a single action. The thoroughness of his work should be sufficient unto the immediate needs—and no more than sufficient. Nicety, finish, refinement, and permanency, for their own sake, are to be avoided. The military bridge should have no beauty except that which is *inherent in utilitarianism.*

In civil construction great weight is properly given to the factor of safety and to the durability or permanence of the structure. In military procedure these considerations have far less weight. The factor of safety need seldom be as great, although this will depend to some extent on the situation. Where a military force is entirely dependent on a single line of supply, the bridges on this line should have a factor of safety as great or nearly as great, as would be employed in civil practice. But ordinarily, in the combat zone at least, the saving in time and material which results from using a factor of safety of 2 instead of 4 or 5 will more than compensate the risk involved in the possible collapse of structures. The risks attendant upon military operations are so numerous and so great that the slight additional risk of a small factor of safety is of minor significance.

Permanence is of negligible importance in military construction, inasmuch as the structures will never be required beyond the duration of the war, and frequently for much shorter periods. It is generally good economy first to meet the exigency of the moment, and later to repair, strengthen, or even replace the structure, should this be necessary by reason of continued need. The time for which any military structure will be required is usually short, and always uncertain, so that it is not good economy to look too far into the future.

Warfare is an economic art, no less than any of the pursuits of peace. In war we have a mission to perform, which is the achievement of victory; and this mission should be accomplished with the least possible expenditure of blood and treasure—in other words, in the most economical fashion.

This demands that victory be won in the shortest possible time, since the cost of war is nearly proportional to the duration of the conflict. Moreover, in the prosecution of the conflict itself, victory comes to the combatant who can most rapidly mass his resources of men and materials at the critical points. It is for these reasons that time is so important an element in all military operations, including construction.

Waste not Justifiable

The achievement of our purpose in war calls for a rapid expenditure of both life and material. It is the duty of the Commander, however, to expend in the most economical manner possible the resources which the nation by painful sacrifice has placed at his disposal; and the same obligation rests upon all his subordinates. But great expenditures to accomplish great results are not wasteful.

The engineer must thoroughly disabuse his mind of any belief that military necessity ever calls for or justifies waste of the nation's resources; but he must not allow a penny-wise inclination toward economy of material or money to cause him to overlook the greater necessity for economy of *time*. Material should be freely expended to save time, but it should never be wasted. However great the resources of the nation, there is always a dearth of construction material in war, due not alone to lack of material but also to lack of transportation facilities for delivering it at the places where it is required. A reckless use of material in one locality may mean the failure of important operations in some other place where the material thus wasted is needed. A small saving of material must not be made at the expense of a great waste of time; but the soldier who deliberately or carelessly wastes any useful material is guilty of highly unpatriotic, not to say criminal, conduct.

Classes of Military Bridges

There are two general classes of military bridges;

(*a*) Those constructed in rear of the battle lines, in the zone of communications, not in the immediate presence of the enemy.

(*b*) Those constructed at the front, within the combat zone or the area subject to hostile fire and raids.

Structures of the first class are erected for purposes and under conditions approximating those of civil construction in time of peace. The need for such bridges is determined mainly from strategical considerations, and they seldom have any intimate relation to the tactical operations of the combatant forces. As compared to bridges within the zone of tactical activities, they are usually needed for a relatively long period—from several months to the duration of the war. They are required to carry heavy loads. They may be constructed by non-combatant troops, by hired civilian labor, or even by contract. Construction plant, similar to that employed in civil works, will be utilized whenever it is available. With the exception

of the facts that they are less permanent in their nature, and that no consideration is given to the appearance of the finished structures, these bridges are built in accordance with the recognized rules of good civil practice.

Structures of the second class, those at the front, are erected to meet the immediate tactical requirements of the combatant forces. They are usually in the form of hasty makeshifts of a crude character, and are built by combatant troops in accordance with the economic principles of warfare heretofore enunciated.

There are no sharp lines of demarcation between civil practice and military bridges of the first class, nor between military bridges on the lines of communication and those at the battle front. Military economics are occasionally applicable to civil practice, and vice versa.

Types of Military Bridges

The particular type of bridge to be employed in any situation depends on the nature of the stream—its width, depth, swiftness of current, and liability to flood; the character of the approaches; the labor, plant, and materials available; and the loads to be carried. Every type of bridge known to civil practice has been employed for military purposes, including pile and framed trestles, and cantilever, truss, girder, suspension, floating, and arch bridges. The last mentioned, either of steel or masonry, are very rarely used, and only on the lines of communication. Military bridges are, in general, crude, impermanent, and makeshift forms of their civil prototypes.

Any bridge for which the erection requires a long time and an elaborate and heavy plant will usually be avoided in military practice, and is out of the question for any tactical purpose. The considerations calling for such bridges in civil practice will usually have much less weight in time of war. For example, a long-span-truss or cantilever bridge may be adopted in order to avoid the expense of very deep foundations, to evade danger from floods, or to meet the requirements of navigation. Deep foundations are out of the question in emergency military bridging, and are evaded by the use of the floating equipage, by portable (sectional) trusses, by long-span suspension-bridges for very light loads, or by ferries for occasional traffic too heavy for the bridges. Danger from flood is avoided in a similar manner; but, as the period for which a military bridge will be required is always relatively short, it will frequently be better to run the reduced risk of flood damage, rather than spend much time in guarding against it. If the flood risk be really great and imminent, it is best avoided by using the floating equipage, unless a clear span of moderate length will meet the situation. In war the rights of civil navigation must give way to military necessity. Military traffic, both over the bridge and along the stream, may be regulated so as to interfere with each other as little as possible, draw spans being provided, if necessary, for the passage of vessels.

The Typical Military Bridge

The typical military bridge, therefore, since it must be erected in a short time and without elaborate plant, will be characterized, as a rule, by shallow foundations, by a relatively large number of piers or supports with correspondingly short spans, and by structural members of small size, light weight, and great simplicity. If we add that timber is the material most frequently employed, this delineation of the typical military structure will be recognized as a description of the short span, framed-trestle-and-stringer, timber bridge; and, in fact, this is the type employed in the majority of cases in military practice. If we now permit the occasional use of pile trestles instead of framed trestles, I-beams in place of wooden stringers, and simple wooden or sectional steel trusses for greater spans, and include the standard floating equipage, we shall have enabled the military bridge builder to meet *nearly all* situations with which he will be confronted.

Some of the economic features of the more usual types of military bridges will now be considered.

Framed Trestles

The framed trestle requires no plant, other than simple tackle, for its erection. It is readily constructed from a great variety of materials, and by unskilled labor; and it meets the majority of emergency situations. The framed trestle, as we have seen, is accordingly the favorite type of support in hasty military bridging in the combat zone. The usual form of bent is the simple, one-plane type with cap, sill, two or more posts or legs, and diagonal sway-bracing.

The trestle will be stiffer against lateral stresses, if the outer posts be inclined or battered; but with unskilled labor it is easier to make all the posts vertical. If pieces of sufficient length are available, the stiffness may be greatly increased by extending both the cap and the sill a foot or more beyond the outer posts and attaching the sway braces to the ends of the cap and sill as well as to each post.

If the depth of water is such that the trestles tend to float up, the bottoms of the posts may be boxed in and the compartments filled with stone.

If the river bottom be of low bearing power, a sill of greater width than that of the posts may be employed. The bearing power of the bottom may be increased by brush mattresses or fascines, wooden mud-sills (where the water is shallow), or rip-rap. It is usually well worth while to resort to such measures, in order to avoid the use of piles.

Economic Span of Trestles

The proper economic span of pile or framed trestles is determined by balancing the time and material required for the bents against that required for the stringers. The problem cannot be solved with mathematical precision. It depends upon the height of the trestles, the difficulty of placing

the bents, the load to be carried, the materials available which are suitable for bents or stringers, respectively, etc. Crooked stringers are an unmitigated nuisance; and in a hasty bridge, when only a scrubby growth of timber is available, it may be impossible to secure reasonably straight pieces, except in very short lengths. In such a case the spans must be short. On the other hand, if bents must be built of round timber, and if good dimensioned material is available for stringers, the number of trestle bents may be reduced and the span-length increased. If the loads to be carried are moderate, longer spans will, of course, be permissible. The engineer must scan the situation and come to a decision based on his experience and common sense.

However, it is possible to state the usual limits of good practice. Under average conditions the proper economic span of hasty, military trestle-bridges is from 10 to 15 ft., averaging about 12 ft. For heavy standard bridges, designed to carry the greatest loads of the army, the maximum practicable span, when wooden stringers are employed, is 16 ft., which length requires 16-inch stringers. If I-beams be employed, this span may be increased to 22 ft. For any length over this, either strutted beams or some form of truss should be adopted. When very tall trestles are required, it will often be good economy to increase the span by using simple trusses, or by strutting.

In practice, the number of bents and the span of trestles are frequently determined by the material actually available for stringers. These must be able to carry the required loads; and, whenever possible, they are furnished in standard sizes and lengths for this purpose. Where the material for stringers has been cut to a given length, the span of the trestles is, of course, thereby fixed.

For moderate loads, tall trestles may be built in single stories, if material of suitable length is available. For the standard heavy trestles the following rules have been adopted:

Up to 16 ft., one-story bents with one-story bracing.

From 16 to 24 ft., one-story bents with two-story bracing.

Over 24 ft., two or more stories.

The need for longitudinal bracing between bents depends on the height, the span, and the nature of the traffic. If the height of the trestle exceeds 7 ft. and the span is greater than 10 ft., longitudinal bracing should be placed in each alternate bay. For considerable heights it is well to brace all bays, if practicable. In multiple-story bents each story should be separately braced.

Spar-Bridges

The term spar-bridge is a general designation for a military structure of rough (round) timber. Such bridges are built from necessity when better material is not available. They will continue, as in the past, to be a characteristic type in the operations of relatively small and poorly equipped

forces, especially in sparsely settled countries under bad conditions as to transportation.

Because of lack of other fastening materials, spar-bridges are often assembled by lashing the trestles. In such cases it is impossible to use "rider" sills and caps, placed on the tops and bottoms of the posts; hence "ledgers," attached a short distance below the tops and above the bottoms of the posts, are employed. Stringers are also lashed, and decking is secured by means of side-rails placed over and lashed down to the outer stringers. The term "spar-bridge" is generally used to describe such a characteristically military structure. Whenever possible, however, even bridges of round timber should be roughly framed, have rider caps and sills, and be fastened with bolts, spikes, or dogs.

A special form of spar-bridge, known as the lock-spar, is a structure in which the trestle bents are tilted towards each other and locked together or to a frame placed between two bents (double lock). These are of very limited application, but may occasionally be used to advantage in spanning a deep gorge. With the double lock the bridge is practicable in spans up to 50 ft. for moderate loads.

Spar-bridges, especially when lashed, if of necessity employed in the first instance for a hasty crossing, should be replaced promptly by a better type of bridge, as they are unsuitable for continued heavy traffic.

Pile Trestles

Framed bents, as they are so easily erected without plant, will be generally employed when the bottom is firm enough to support them, and when scour can be prevented. When the bottom is of very low bearing-power, or subject to scour, as in the case of soft mud or shifting sand, piles will be required. Pile bents have the disadvantage of necessitating longer pieces, more time, and the use of a pile driver for their erection. However, they are required in many situations, especially for railroad bridges, where even a slight settlement, such as might result from the use of framed bents, would be dangerous. Accordingly, a number of efficient portable pile-drivers should form part of the equipment of an army; and, when such are available, pile trestles will often be preferred to framed ones. Pile-trestle bents are subject to the same economic considerations as framed-trestle bents, except that, being stiffer, they require less bracing.

Trusses

For military purposes the truss has all the economic advantages it possesses in civil use. It is employed to reduce the number of trestle bents, or other type of piers, when the construction of the said piers is, for any reason, particularly difficult. Trusses are also used to give greater clearance as a measure of flood protection, or to span deep chasms or streams without intermediate supports.

Military trusses must usually be erected without falsework, and with

the aid of only animals, simple tackle, and gin-poles or A-frames. There will be difficulty in transporting long pieces for such trusses. Where they must be constructed in the combat zone, the span, consequently, is limited to 40 or 50 ft. in the usual case. Larger trusses may occasionally be built in place; and when plant is available, as in the rear areas, much greater spans will be practicable.

Improvised trusses must be constructed of wood, preferably sawed material, and steel tie-rods or, in exceptional cases, cables. Accordingly, simple triangular (King post), Howe, and Pratt trusses (generally erect or through) will be the usual types. Where only light material is available, lattice or bow-string trusses may be employed. These types should be built as nearly as possible in accordance with civil practice in similar structures. Joints should be as simple as practicable, requiring a minimum of expert carpentering. Round timbers may be used in the simpler forms of trusses, if dimensioned stuff is not available. Deck spans may be employed for reasons similar to those which would dictate their adoption in civil practice.

Sectional Trusses and Girders

Sectional, portable, or "knock-down" trusses and girders, in wood and especially in steel, were the principal development in military bridging during the World War. They were demanded by the augmented density of traffic, and especially by the tremendous increase in military loads, which now include heavy motor-trucks, artillery of great calibers, tractors, and 30-ton tanks. The improvised bridges, long characteristic of military operations on a smaller scale, were, alone, inadequate to meet the unprecedented situation. It was recognized that there was needed some form of standardized superstructure, easily transported, rapidly erected, adaptable to varying spans, and capable of carrying the heaviest loads. This demand was best met by portable, sectional, steel girders and trusses; and various types of such structures were developed by the British, French, and American Armies.

As usual in such cases, a great variety of types was evolved; but, in the interests of efficiency, the number of such should be limited, and each should be made adaptable to varying conditions.

For the American Army, sectional steel girders (I-beams) for spans up to 30 ft., and sectional steel trusses, for spans from 33 to 90 ft., were designed. The British employed sectional trusses in spans up to 180 ft.

In order to permit transportation in motor trucks, the truss sections (panels or bays) were limited to 11 ft. 3 ins. Spans from 33 to 90 ft., in increments, could be erected with this equipage. The sections were connected for erection by bolts, no field rivets being used. These trusses can be employed with any type of support or pier. The truss is erected without falsework, being bolted together on one bank, and placed by launch-

ing with the aid of tackle and an A-frame derrick, or by a counter-poise of additional sections of truss.

It is certain that sectional steel girders and trusses will be used on a vast scale in future wars of any great magnitude.

Portable girder-spans, usually of timber, complete with flooring, in one or two sections, are provided for the passage of foot troops and artillery over trenches, ditches, and small shell-holes, in following up an attack. These bridges are transported on combat wagons or artillery caissons.

Cribs

Cribs may be usefully employed for piers on very unstable bottoms, especially when there is a swift current, and for abutments. They have the advantages that they require no plant, and can be built of short pieces of almost any material. Cribs have greater power of resistance to floods, ice, and drift than have simple trestles, provided they are solidly constructed and filled with stone. Cribs are often employed as foundations for framed trestles, the crib-work being carried above ordinary flood level.

Trenches, ditches, shell-holes, and small ravines can be made passable by filling them with any debris that may be at hand.

An interesting development of the World War was a small cube or crib of structural steel, any number of which could be bolted together to form a bridge abutment.

Suspension Bridges

Suspension bridges are occasionally employed in military operations. For spans exceeding 50 ft., when no plant and no sectional trusses are available, they will sometimes meet the situation. For military purposes they have the advantage that the only essential parts are the cables, which are easily transported. If these are at hand, the remaining portions, being small and light, can usually be obtained in any locality. Light suspension bridges are easily erected without plant. When materials for a fixed bridge must be carried with a rapidly-moving column, the suspension type has the advantage of requiring the least material, for a given span and capacity, of any kind of bridge. Moreover, its parts, being small, are easily transported and handled.

The suspension bridge is specially applicable to long spans combined with light loads, but even in such situations the ponton equipage or ferries will usually be preferred for stream crossings. Hasty suspension bridges may be built to carry wagons, but for motor traffic they are unsuitable. In general, they are used only for foot bridges of relatively long span. Their ideal function is for foot traffic and pack transportation over wide and deep ravines in mountainous country, where the ponton equipage or sectional trusses are not applicable.

Because of the difficulty of handling in the field, cables are generally

limited to one inch diameter, as many as necessary being employed. Standing trees are utilized as towers and anchorages when this is feasible.

It will usually be impracticable to insure vertical reactions on the towers; and as roller bearings are not employed, the said towers should be of the sawhorse-trestle type and well braced, in order effectively to resist the overturning moment. The sag of cables usually employed for military bridges is from $\frac{1}{10}$ to $\frac{1}{7}$ of the span. Oscillation and undulation are limited by the usual methods, such as lateral bracing of the roadway, trussing the handrails, drawing the cables together at the center, guying, etc. While materials for light suspension bridges, excepting only the cables, may usually be obtained locally, erection will be greatly facilitated if adjustable suspension rods (slings or hangers) are provided in advance.

Ponton or Floating Bridges

The construction of any type of fixed bridge is at best a slow process. Indeed, in the case where an army with heavy artillery and trains is confronted by a wide and deep crossing, the construction of a fixed bridge might require weeks, even months, of time—in fact it might prove utterly impossible under some field conditions, for instance, in the absence of heavy and elaborate construction plant. Tactical requirements cannot brook such delays; and, to meet situations of this kind, some form of portable bridge, with floating supports and capable of extremely rapid installation, is absolutely indispensable. Accordingly, all modern armies are equipped with such bridges, which are known as ponton or floating equipage.

The floating equipage which up to the present has been used in our own army (except the foot-bridge) was devised prior to the Civil War, and has been employed with conspicuous success since that time. It is a tribute to the wisdom of those who designed it that, in over 60 years, no radical changes in the equipage have had to be made. This standard equipage is of the utmost simplicity, consisting merely of any number of boats, called pontons, which are anchored in position and connected by stringers or "balk," resting on the gunwales of the boats, on which stringers the deck planks or "chess" are laid, the whole being secured by lashings. For shallow portions of the stream, near the banks, portable, collapsible trestles take the place of the boats.

There are three forms of ponton equipage, a heavy wagon bridge, a light wagon bridge, and a foot-bridge, the latter devised during the World War. The heavy pontons are of wood, having an available supporting power of $9\frac{1}{2}$ tons each. The light pontons are collapsible wooden frames, covered with water-proof canvas, and have a supporting power of 6 tons each. The portable foot-bridge employs miniature canvas pontons and provides a path 2 ft. wide.

The advantages of this type of bridge are its portability and the extreme rapidity with which it can be installed. The approximate weights of the material per running foot of bridge are as follows: heavy equipage, 170 lbs;

light equipage, 130 lbs.; foot-bridge, 16 lbs. One "division" (225 ft. of bridge) of the heavy equipage requires 16 wagons to transport it, and covers a road space of about 300 yards. One division (186 ft. of bridge) of the light equipage requires 14 wagons and a road space of about 250 yds. One division (285 ft. of bridge) of the foot-bridge can be transported in one three-ton truck.

Lacking the standard equipage, almost any kind of boats, rafts, casks, or in fact anything that will float, may be utilized as supports for an improvised bridge.

The heavy equipage has the advantages of greater strength and capacity, and less vulnerability to hostile fire. The light (canvas) equipage has the advantage of greater mobility in transport. The heavy train is required when heavy loads are to be carried, when very swift streams are to be crossed, or when the bridge must resist ice and drift, or stand up under hostile fire.

The erection of the ponton bridge takes the form of a drill, and is accomplished in an incredibly short period of time by men who have been properly instructed. The equipage, moreover, is so simple that average soldiers may be quickly trained to install it.

There is no stream too wide, too deep, or too swift for the ponton equipage, when handled by trained men. As an indication of its capacity, adaptability, and speed of erection, the following historical examples will be of interest.

On June 15th, 1864, Gen. Grant, in his attack on Richmond, had need of a crossing of the James River. The stream was deep, and so swift that the pontons could not be held by their own anchors, it being necessary to attach their cables to schooners placed in the stream. The crossing was over 2,000 ft. wide, 101 heavy pontons being employed, and was completed in $5\frac{1}{2}$ hours, or at an average speed of 6 ft. per minute.

In February, 1862, a bridge of 60 boats was thrown across the Potomac at Harper's Ferry. The river was in flood—a perfect torrent—carrying great quantities of ice and drift. In spite of all these difficulties, the structure was successfully completed in 8 hours—average speed of erection, $2\frac{1}{2}$ ft. per minute.

In the winter of 1919 a ponton bridge was built across the Chattahoochee River at West Point, Ga., by a detachment of the 7th Engineers with civilian assistants. The bridge was reinforced to carry heavy traffic, and the flooring was spiked in place. It was installed during a flood, the current varying from 5 to 10 miles per hour, and the water level rising 4 ft. during construction. The length was 440 ft.; and the bridge was completed in 10 hours' working time.

These performances under service conditions, remarkable as they be, are quite eclipsed by the exhibition bridge constructed across the Rhine (near Honningen) by the 1st Engineers, American Expeditionary Forces, during the military occupation of Germany (1919). The river at this

point is 1,450 ft. wide, with a maximum depth of 25 ft. and a current of 3 to 4 miles per hour. The bridge was built from both ends, 400 trained men being employed, and 93 pontons (of the German equipage) being used. The structure was completed in the astonishing time of 41 minutes 8 seconds, or at an average rate of more than *thirty-five feet per minute*, the world's record for speed in bridge construction.

No other type of bridge ever devised is capable of anything approaching such speed in erection. By contrast with the time required in the cases mentioned, one of the best examples of the construction of a heavy, military trestle-bridge was a structure built over the Little Pedee River in the Civil War, in which a length of 100 ft. was completed in about 9 hours.

The ponton bridge is designed to meet emergencies; and if the crossing is to be required for any considerable period, the ponton equipage should be released by the construction of a fixed bridge, in order that it may be available for fresh emergencies. For the requirements of a moving column in a theatre where stream crossings are encountered, the ponton equipage is indispensable. In addition to its use for bridging, the equipage may also be employed for ferrying, as in the passage of a stream by force in the face of the enemy.

If it be necessary to employ the ponton bridge for a considerable period, the chess (floor plank) should be protected by a sheathing of thin lumber or by hay or brush. These chess are very thin ($1\frac{1}{2}$ in.), and, if unprotected, would soon be worn through and ruined by continued traffic.

As just stated, the floating equipage, if available, will often be the first solution for an important emergency crossing. Subsequently, and as soon as possible, it should be replaced by a trestle bridge, which in turn may later be used as falsework for a more elaborate structure.

The adaptability of the ponton equipage is very great; it is ideally fitted to meet military emergencies; and its value as a saver of time cannot be over-estimated.

New Types of Ponton Equipage

The old types of ponton bridge, which have so long served military needs, and which are of generally similar design in all armies, like other former types of military bridges are inadequate to the needs of modern traffic. In future wars a heavier equipage will be required, and even this must be capable of "reinforced" construction to carry the augmented loads. Such an equipage has been designed for the American Army. The pontons have an available buoyancy of 10 tons, the span has been decreased to 15 ft. and heavier balk are used. An all-metal boat, or one of metal sheathing on a wooden frame, will be employed. The adjustable trestles for end spans will be of steel. This bridge will normally carry a concentrated load of 13,500 lbs. on one axle, and may be reinforced to carry an axle load of 20,000 lbs. This will take all the loads of a corps or army, excepting only heavy tanks and the guns and tractors of the artillery weighing

more than twenty tons. The reinforcement consists of an intermediate roadway-bearer supported by the heavy side rails. The wagons used to transport the new equipage will be such that they can be hauled by animals, truck, or tractor.

Railroad Bridges

In the construction of railroad bridges, military practice follows civil practice more closely than in highway bridges. As they must usually carry heavier loads, and as variations in either horizontal or vertical alignment are much more serious, greater attention should be paid to the solidity of the structure, including its foundations. Pile trestles, girders, and trusses on piers of pile clusters are the usual types of military-railroad bridge. Most of them, except for light, narrow-gauge railways, will be in rear of the combat zone, though often bridges for standard gauge must be constructed close to the front.

Foot-Bridges

Foot-bridges, employed usually to meet tactical emergencies, exhibit a greater variety of forms, and have heretofore been less subject to standardization than either vehicle or railroad bridges. In a crisis, any design and any material that will serve, or even partially serve, the purpose must be employed. Trees, cut near the bank and allowed to fall across the stream, often have enabled a combatant force to meet a grave emergency. As traffic accidents thereon will be less serious than in the case of other bridges, less attention is usually paid to the factor of safety. If this be very uncertain, the bridge should be tested by sending a few men across, and the traffic should be regulated to prevent crowding.

The demands for the rapid passage of foot troops over streams are so frequent and insistent in modern warfare, that it is certain that in future, even more than in the past, standardized, portable foot-bridges will form part of the equipment of combatant organizations. A number of such standardized foot-bridges, including the light ponton type heretofore described, were devised during the World War. Floating types will be the ones most commonly employed, these being supplemented by light sectional trusses and, in some instances, by suspension bridges. The floats which have been successfully used include light canvas pontons, casks of wood or steel, rafts of wood or cork, and Kapok Rafts. The latter have the advantage that they are unsinkable by rifle or machine-gun fire.

The width of the roadway or path of foot-bridges is from 2 to $2\frac{1}{2}$ ft. A greater width is unnecessary for the passage of men, and unduly decreases the mobility of the equipage, which is an essential requirement. Such a width (2 ft.) does not permit the passage of machine gun carts, which must seek other means of crossing. Machine guns, light mortars, and one-pound cannon accompanying the infantry, with their ammunition, may be carried across by hand.

Deck or Flooring of Military Bridges

Dimensioned lumber, never less than 2 inches thick and preferably 4 to 5 inches (except for foot-bridges), is most desirable for decking. The construction of a deck of poles is a slow process; because it is a tedious task to collect and prepare sufficient material for a bridge of considerable length. Poles for decking should be at least $3\frac{1}{2}$ inches in diameter. They should be well spiked so that they will not rattle. As a pole deck is usually very rough, it will generally be advisable to chink the openings and cover first with brush and leaves and then with a layer of earth. The additional dead load thus brought upon the structure should not be overlooked.

Woven-brush mattresses, supported on deck poles at relatively wide intervals and covered with earth, make a satisfactory deck for foot-bridges.

The wear on decking resulting from military traffic is very great, hence light material is unsatisfactory. For heavy bridges on important routes a deck 5 inches thick, preferably of hardwood, is now regarded as standard. If only 2-inch material is available, a double thickness should be used. The distance between stringers in *feet* should not be more than the thickness of the decking in *inches.*

Width of Roadway

As the amount of material and the time required for construction increase with the width of the roadway, military bridges are made no wider than necessary, and are generally limited to a single line of traffic, with clearance for the passage of footmen, horsemen, or motor-cycles. As military traffic usually moves in trains and can be closely regulated, it requires a less width of roadway than unregulated civil traffic. For the standard bridges, a roadway of 10 or 11 feet is employed. If it be necessary to provide for the continuous movement of traffic in both directions, two bridges side by side may be adopted; or, if the traffic be very dense, three or four parallel bridges may be built. Two-way bridges are, of course, also employed. A 15 ft. roadway will pass two lines of traffic, but can hardly be called ample for the purpose, a width of 18 or 20 feet being better.

The maximum capacity of military bridges in service should be developed by efficient traffic control.

Side-Rails and Hand-Rails

Side-rails or wheel-guards of ample strength should never be omitted from any vehicle bridge. Hub-guards and hand-rails are generally provided for through-truss spans, including suspension bridges, largely for the protection of the bridge itself; but they are generally omitted in the case of deck spans. On the other hand, side-rails are unnecessary for foot bridges, except to secure the flooring; but hand-rails are generally provided.

Materials Employed in Military Bridging

Any available material must be utilized in the construction of military

bridges. Even bamboo has been successfully employed to carry heavy loads.

Timber

Timber can be adapted to the requirements of bridge construction more easily and rapidly than any other material. It is also the most generally available of all bridge materials. For civil constructions it has the disadvantage of lack of permanence—a consideration having little or no weight in military operations.

Timber, accordingly, is the favorite material of the military bridge builder. Dimensioned (sawed) timber is greatly preferable to round timber (unsawed logs and poles); first, because it can be much more quickly incorporated in the bridge; second, because it makes a stronger and better structure; and third, because it lends itself better to standardized designs and methods of construction. A few standard sizes of lumber (sawed timber), sufficient to meet the requirements of the standardized designs, are preferable to a great number of miscellaneous sizes that are difficult to adapt to any design.

Military bridges must be and are frequently constructed of round timber, but this is from necessity and not from choice. Round timber is seldom used (except for piles or posts of framed trestles), if dimensioned material is available.

Piling

Piles are frequently employed for abutments, trestles, and piers in situations where the nature of the bottom does not permit the use of framed trestles. The use of piles necessitates a pile-driver, and consequently at the front should be avoided if possible, although frequently necessary there. Framed trestles, which require no plant for their erection, are preferable, even if it be obligatory to rip-rap the bottom in order to increase its bearing power, or to prevent scour. Nevertheless there will be situations, even at the front, where piles must be used; and portable pile-drivers will be a part of the engineer equipment of every army.

Steel

Any steelwork which involves field riveting will find little application in military bridging, even on the lines of communication, for the reason that its erection demands special plant and skilled labor. Rolled sections (I-Beams) are frequently employed as stringers in trestle bridges on the lines of communication, and even at the front, when it is necessary to carry very heavy loads, such as tanks.

Portable, demountable, steel truss-bridges, capable of very rapid erection, and designed for both light and heavy traffic, were employed in the World War. The designs were very ingenious, and such structures will undoubtedly be used to a great extent in future.

Special fabricated-steel girders are frequently employed for long stringer-

spans, mainly for railroad bridges on the lines of communication. Their purpose, of course, is to reduce the time required for pier construction. They differ in no essential from civil structures for similar situations.

Concrete

Concrete is utilized for culverts, for foundations of framed trestles, occasionally for abutments of bridges on important routes of communication, and in the repair of old masonry bridges. For piers, piling or crib-work is almost invariably preferable, even in the case of large and important bridges. Arches and girder spans of concrete will seldom be used. In the battle zone concrete finds very little application in bridgework.

Stone

Stone is employed as rip-rap for the protection of piers and abutments, and as crib-filling. Stone masonry will only exceptionally be used for abutments or wing and retaining walls; for there will be but few situations in which either concrete or rubble-concrete would not be preferable. If no cement is available, lime-mortar or dry-stone walls are sometimes employed.

Paint

Paint and other protective materials are sparingly used in military bridging. Ordinarily the time for which the structures will be required does not justify the employment of these preservatives. Steelwork should always receive a shop coat, but field coats will usually be omitted.

Joints and Fastenings

As military bridges must be constructed in the least possible time and usually by unskilled labor, intricate joints, involving expert carpentering, are to be avoided. Generally a plain butt joint, requiring no carpentering, other than squaring the end of a timber, should be used. The fastenings ordinarily employed for hasty bridges include lashings of rope, marline, or wire; dogs; fish-plates or "scabbing"; drift bolts; through (screw) bolts; and spikes.

Lashings are employed in the absence of other materials, or, as in the floating equipage (which is generally lashed throughout), when the bridge is to be later dismantled and the material salvaged. They are not a very secure form of attachment, especially when they actually carry the load, as in the case of the cap of a spar trestle. In such structures it is advisable to dap or notch the timbers together. Marline gives a tighter lashing than rope, but not such a strong one; and small rope is preferable to large. Lashings do not stand racking well, and often are not sufficiently durable even for military uses.

Dogs are easily made of any iron available. They should not be employed to carry a load, but only to hold timber in position.

Drift bolts are commonly employed as fastenings for all parts of a bridge except the deck. They are readily made up in any desired length, and are

quickly driven. A variation in the length of the bolt makes no difference, provided it is long enough. It is not necessary to point the ends. Lag screws may be used, if available.

Through bolts, with nuts and washers, are the most secure form of fastenings; and, if procurable, they should be used for all important joints, except sills and caps which are usually "riders" (butted on the trestle legs), and for which drift bolts, dogs, or scabbing, are employed. For round timbers, through bolts have the disadvantage that they are generally either too long or too short, requiring the use of wooden washers in one case, or dapping out the timber in the other—in either instance involving a loss of time. Turned bolts are invariably used in lieu of field rivets in the assembly of sectional steel trusses.

A full supply of nails and spikes of assorted sizes should, of course, be provided.

Sizes of Individual Members

For bridges at the front, the sizes of individual members must generally be such that they can be readily handled and placed by unskilled labor, with the aid of animals and ordinary tackle, gin-poles, or shears. In the cases when plant is available, heavier pieces may be used to advantage. The principal considerations limiting the size of individual pieces are the requirements of transportation, by motor truck as well as by rail.

Plant and Tools

The military bridge builder in the past has made comparatively little use of heavy construction plant in his operations, but this resulted from necessity rather than choice. Construction plant is utilized whenever practicable; and in view of the great capacity now required of military bridges, portable plant is necessary even in the combat zone. However, in the tactical operations of the combat troops the opportunities for the employment of plant will be relatively few. The work of the military bridge builder is spread over a considerable area; he moves rapidly from place to place; and he cannot be unduly hampered in his movements by the necessity of transporting heavy construction plant. Usually the plant is not at hand when needed, and it can seldom be depended upon. Moreover, one of the principal uses of plant is to save manual labor, and this consideration has less weight in military than in civil constructions.

There are, however, a number of pieces of light, portable construction-plant that may often be advantageously employed, even at localities quite close to the front. These include small hoists, pile-drivers, compressors, pneumatic tools, concrete mixers, and rock crushers. Gasoline is the best motive power, because of the compactness of both machine and fuel. For works on the lines of communication, heavy, standard construction-plant is frequently adopted. For the future, portable hoists and pile-driving outfits will be provided for corps and army engineer-troops.

The tools used in military bridge work, like the structures built therewith, should be of the utmost simplicity. There should be an ample supply of the common tools, such as picks, mattocks, axes, saws, hammers, wrenches, shovels, cant-hooks, etc., with a few of the less usual tools that will occasionally be required. All should preferably be of commercial sizes; but, to meet emergencies, miniature tools, easily transported on a pack mule or on the person of the soldier, should also be available in case of need. There will arise situations in which the entire equipment of the bridge builder must be carried on pack mules.

Class of Labor Available

Military bridges must usually be built chiefly by unskilled labor. In every engineer organization there will be a number of skilled artisans of every class; but the majority of the men in the ranks of these organizations, as well as those from labor battalions and working parties from the infantry, will be of the class known as "unskilled labor." This fact has an important bearing on the design and methods of construction of military bridges.

Improvisation and Standardization.

The difficult situations encountered, the varied circumstances under which work must be performed, the emergencies constantly arising, and the necessity of adapting to his purposes whatever materials are available, will require the military bridge builder to be skilled in improvising structures to meet these conditions. Improvisation, in fact, is characteristic of all the operations of military engineering. It is a valuable means of developing ingenuity and resourcefulness; nevertheless it is a practice which results only from necessity, and it has the disadvantages that it consumes valuable time and that it tends towards lack of uniformity both in training and in actual construction.

Standardization, including standard designs, standard materials, and standard methods of construction and of training, is at least as useful and economical in military practice as in civil practice. It has the following important military advantages:

(*a*) A standardized structure can always be erected in less time than one in which extensive improvisation is necessary, especially by troops who have been trained in standardized construction. The saving of time results from both the fact that it is possible to use standard plans, and that loss of time from improvisation is eliminated.

(*b*) A further saving of time is effected by the fact that standardized materials may be prepared beforehand at the engineer depots, and sent to the front as needed. Lumber can be cut to exactly the required dimensions, bolts will be of exactly the right length, and special pieces of all kinds can be made up in rear, so that it will not be necessary to spend time in preparing them under difficult conditions at the front.

(*c*) Standardized structures require less skilled labor than improvised structures.

(*d*) Standardization promotes uniformity in training and practice.

(*e*) A standardized structure is nearly always superior to an improvised structure in every item of utility.

(*f*) In a hasty, improvised bridge constructed of random materials, the actual strength of the structure is usually a matter of uncertainty and guess-work. In standardized designs the safe loading is accurately known, and the chance of accident from overloading the structure is correspondingly reduced.

The benefits of standardization being so manifest, standardized-type plans have been prepared for all forms of military bridges, even for those constructed of local materials. Standardized material for these structures is prepared in the engineer shops, so as to reduce as far as possible the amount of labor required in the field. Standardized designs are, or should be, of the utmost simplicity and flexibility. They should be simple, because only the simplest structures can be successfully and promptly erected under the difficult conditions incident to warfare, involving lack of time, adequate tools and plant, skilled labor, etc. They should be flexible, because no two situations are exactly alike; and the type plans must be modified in practice to meet the actual conditions. It should be possible, with a few minor variations, to adapt the type plans and the standard materials to any situation, with a minimum of improvisation. The best example of standardization in military bridging is the sectional steel truss heretofore described.

It has been said that standardization has a tendency to discourage invention, and hence is inimical to progress. While it must be admitted that there is some truth in this contention, the tremendous results that have been achieved through standardization, in both civil and military practice, are a sufficient justification for its adoption. There will be an ample field for the inventive talents and resourcefulness of the engineer, both within and without the limitations imposed by standardization. Moreover, for every inventive genius who has been discouraged by standardization there are thousands of average men who have profited by it; and it is the average man with whom we invariably reckon in military operations of all kinds. Standards should be revised and improved in the light of experience; and this was frequently done during the World War.

Because of the density of traffic and the heavy loads incident to modern military operations, standardized bridge designs will be employed more frequently in the future than they have been in the past. There were times during the World War when the regular daily traffic on a single road was 5000 motor trucks (in both directions), and during troop movements this reached a maximum of more than 17,000. Rough, improvised structures would not be adequate to the needs of such a traffic.

Utilization of Existing Bridges

To save time, material, and labor and to obtain quicker results, existing bridges should always be utilized as far as possible. Unless the bridge is a complete wreck, it will nearly always be easier to repair or strengthen it than to build a new structure. Existing bridges should be examined by an engineer who is familiar with bridge construction in the particular locality. Any bridge built on approved standard design, and the parts of which are in good condition, may be regarded as safe for its rated loading. Any bridge that exhibits a radical departure from standard design should, of course, be looked upon with suspicion. The horizontal and vertical alignments of the structure should be noted, after which all its parts should be examined for weakness or deterioration. The conduct of the bridge as loads pass over it should be carefully observed.

A weak bridge of any type may be strengthened by placing additional trestle bents between its supports, or by strutting its stringers. All bracing should be thoroughly reinforced. Even in the case of a demolished bridge, it will generally be possible to utilize at least its abutments and piers, or what remains of them. A fallen truss may often be raised to position and its undamaged portion made available by introducing a new pier. A few hours' work will often render a weak bridge sufficiently strong to carry military loads; whereas, if it be allowed to break down, extensive reconstruction or even a new bridge may be needed. It certainly is a very poor bridge indeed that cannot be strengthened sufficiently to carry the desired loads, with less labor than would be required to build a new structure; and, in the interests of economy, such strengthening should always be undertaken.

Utilization of Local Resources

To save transportation, to promote mobility, and to meet the frequent emergencies in which no standard materials will be available, the military engineer will often have occasion to build his bridges of any local materials that may be available. Structures built of such materials are generally known as hasty or improvised bridges. While the utilization of local resources may save transportation, it will not economize time in construction. A great deal of time and effort must usually be expended in finding and collecting the available material, and in transporting it to the site of the work; and when it is there, further difficulty will be encountered in adapting the heterogeneous collection to the standard designs, or indeed to any design

In a wooded area, round timber will be available, and will prove to be very satisfactory for the construction of bridges, though by no means equal to sawed lumber of standard sizes. As before stated, the military engineer does not use round timber from choice, although it may be remarked that bridges constructed of such material have a characteristically military appearance, and sometimes a certain rustic beauty quite pleasing to the

eye. But it is also to be remarked that it was to avoid the loss of time involved in laboriously collecting the material for and constructing such bridges, that the ponton equipage and the standard types of fixed bridges were devised. The construction of a bridge of round timber is a rather entertaining recreation, and engineers occasionally overlook the loss of time thus involved; hence it should be avoided whenever possible. The collection of the material and the laying of a pole deck, in particular, constitute a time-consuming operation. In building a bridge of mixed material, the round timber may be employed for trestle posts, sills, and caps, and, if necessary, also for stringers and side rails, leaving the dimensioned stuff for bracing, stringers, and deck—especially the deck.

In almost every town, unless it has been previously overrun by the military, a small stock of lumber can be found; and frame buildings may be wrecked, if necessary. This, in combination with standing timber, will often meet the needs of the situation. Local materials are, in general, decidedly inferior in all respects to standardized materials from the engineer depots. They must, however, frequently be employed; and the engineers should be skilful in collecting them and in adapting them to their needs.

Loading of Military Bridges

The military bridge builder should be familiar with the usual loads that bridges in various situations and for various purposes will be called upon to bear. Usually these loads are quite definite, and the standardized designs are based upon them. As the factor of safety employed in military bridging is almost always low, especially at the front, vehicles heavier than those for which the structure was designed should not be permitted to cross it. Nevertheless, it is generally impossible to foresee the extent to which any bridge may be used. Bridges on important routes of transport are almost certain to be called upon to carry the heaviest loads. Drivers of heavy trucks will proceed across any bridge they come to, whatever warning signs may be posted, unless a guard is placed at the structure.

Accordingly, it is desirable that, on any main route, or any road apt to become an important route, the bridges, whatever their immediate purpose, should be built to carry the heaviest military loads. The supports, at least, should be made of ample strength, which will allow the bridge later to be reinforced to carry heavy loads by introducing additional supports or otherwise strengthening the superstructure.

In general, the first crossing will be a hasty improvisation, or a light foot-bridge of some standard design. If the need for the crossing continues, this bridge may be replaced by the ponton equipage, and subsequently by an improvised or standardized fixed bridge. By this time the probable future needs of the route will be known with some certainty, and future steps can be taken accordingly.

There should be an ample number of through routes having bridges capable of carrying the greatest loads, including tanks and heavy artillery.

These routes for heavy traffic should be properly sign-posted; and information concerning them should be circulated throughout the command. It will evidently be impossible to provide all routes with the heaviest bridges; hence a rigid traffic supervision should be exercised to prevent heavy vehicles from wandering into the minor routes where the bridges have not sufficient strength to carry the loads that would thus be imposed upon them.

The foregoing requirements conflict to some extent with the previous statement that a hasty bridge should be built with a low factor of safety and to meet the exigency of the moment only. Most bridges constructed for tactical uses will be off the main routes of transport. In any case, the engineer must decide whether the probable future service justifies the expenditure of more time in initial construction.

In the operations of the large, fully-equipped, modern army, standardized steel superstructures, capable of carrying its heaviest loads, will hereafter be characteristic of all important routes of transport, even quite close to the front. In the operations of small, detached, rapidly-moving, and lightly-equipped forces in a sparsely settled country, the lighter forms of floating bridge, and various kinds of improvised structures to carry moderate loads, will in the future, as in the past, be typical of bridging activities. In any campaign, improvised structures will always be required in the combat zone.

Transportation of Materials

There is always a shortage of transportation for every purpose in time of war. Such important supplies as ammunition, weapons, and food must naturally take precedence over construction materials. When the necessity for a bridge can be foreseen for a reasonable time, it may be practicable to assemble in advance the plant and material requisite for its erection. This will often be possible on the lines of communication; but at the front it will be absolutely impracticable to have on hand, at *every* locality where an emergency may demand a bridge, all the materials required for the standardized design. The engineer is thus compelled to improvise and adapt to his needs such miscellaneous material as he can secure on short notice, locally or otherwise.

Where military activities can be foreseen, depots or "dumps" of construction material are assembled in the vicinity, so that only local transport will be required. Large, heavy, awkwardly-shaped pieces will always be delayed in transport. They are difficult to load and ship; moreover, they arouse the hostility of the transport personnel, who have a natural tendency to push them towards the bottom of the "priority list"; hence, all bridge material should be of a size and weight that will insure reasonable promptness in transportation.

While it will seldom be possible to supply the engineer with *everything* that he needs, it will often be practicable to provide certain materials that

will greatly reduce the time required for construction. For example, the building of a pole floor is the slowest item of an improvised bridge. If dimensioned flooring material (decking) alone can be sent up from the rear, a material loss of time is eliminated, and the engineer will be able to improvise the rest of the structure. Bolts, spikes, wire, cordage, etc., form a very small part of the weight of the bridge—they come in small packages and are easily transported. Without them the time and difficulty involved in building the structure would be greatly increased; hence, these fastenings, at least, should be made available wherever needed.

Selection of Site

In many cases there will be little room for choice in the selection of a site. New bridges must frequently be built at localities where former bridges have been destroyed. In any case *accessibility to adjacent roads* on both sides will be a controlling factor in fixing the site, except for footbridges. The location must fulfill the tactical requirements; but, as a rule, the chief tactical requirement is rapid construction, and, accordingly, the site is selected with this in view. Where there is room for choice, the following conditions, affecting the time required for construction, will be given consideration.

(*a*) *Width of Stream.* This, of course, directly affects the length of the bridge and the time required for its construction. Other things being equal, the narrowest point is preferred.

(*b*) *Depth of Water.* The time and effort required for the construction of any type of fixed bridge increases rapidly with the depth of water, unless the stream can be crossed with a single span (truss or suspension). For the ponton bridge there should be sufficient depth to float the boats. Any greater depth does not increase the time required, except that very great depths make the anchorage of the pontons somewhat more difficult. Its independence of the depth of water is one of the principal advantages of the floating equipage.

(*c*) *Swiftness of Current.* A swift current greatly increases the difficulty of constructing and maintaining any type of bridge, except one with a single span from bank to bank. The slowest current is found in wide, straight reaches of the stream.

(*d*) *Nature of Bottom.* A very soft, yielding bottom will demand pile or crib construction, whereas a hard bottom will support framed trestles, which can be more rapidly placed. A ponton bridge, or a fixed bridge of a single span, is, of course, independent of the nature of the bottom, as it is of the depth.

(*e*) *Nature of Banks and Approaches.* A low, flat, and marshy approach, or one subject to overflow, will demand either a very long bridge, or a causeway, the construction of which is apt to require much more time than the bridge itself. On the other hand, high banks will necessitate tall

trestles; or else they must be cut down to provide approaches with a practicable grade—an operation usually involving much labor. The ponton bridge demands low approaches, as its height above the water surface cannot be increased. For crossings of small or moderate width that can be cleared with a single span or by using a small number of intermediate supports, high banks will be no great disadvantage; consequently, for any type of bridge, the most favorable crossing is one having low, firm banks of equal height, but above the level of any flood likely to occur during the time that the bridge is in use.

(*f*) *Materials Available.* If the bridge is to be constructed in whole or in part of materials obtained at the site, the availability of such local materials, without an excessive amount of transportation, is important. There are certain other considerations, of a tactical nature, that have a bearing on the selection of a bridge site. Amongst these may be mentioned:

(*g*) *Defensibility.* It is desirable that the bridge be located where its construction cannot be interfered with by the enemy, and where it and the troops crossing it will not be subject to artillery or rifle fire after its completion. If built in the near presence of the enemy, it will be advisable, before beginning construction, to ferry some troops across, in order to prevent the enemy's snipers from interfering with the work. It is desirable to locate the bridge in a position where it is sheltered by woods or hills from hostile fire, and where it cannot be seen for any great distance either up or down stream.

(*h*) *Secrecy.* If the crossing is to be made secretly as a surprise, it is desirable that there be a sheltered locality on the near bank, at or close to the bridge site, where the material can be assembled without the enemy's knowledge.

To sum up then, the desirable requisites for a bridge site are: accessibility to adjacent roads; narrow width of crossing; moderate depth and swiftness of current; solid bottom; favorable approaches; usually low, firm banks of equal height; materials, as needed, available near at hand; and a sheltered, concealed, and defensible locality.

In making a reconnaissance for the selection of a site, these various desiderata must be balanced against each other in coming to a decision; for it will seldom be practicable to find a location that combines all the advantages mentioned. In the selection of a site for a railroad bridge there will, of course, be less latitude than in the case of a highway structure. It is to be remembered that the thing sought is not a site for a new bridge, but a *practicable crossing* of the stream. A bridge should not be built if a suitable ford, or an existing bridge which meets (or may be made to meet) requirements, can be found. In anticipation of their probable future use, all existing bridges within range of the army's operations should be promptly siezed and guarded, in order to prevent their destruction by either the enemy or his sympathizers.

Protection against Flood and Drift

Military bridges are protected against flood, ice, and drift by the usual methods of civil practice. Trestles may be strengthened against lateral thrust by guying upstream or strutting downstream. One or more wide spans, with booms or guide walls, may be provided for the passage of ice and drift. Cribs or clumps of piles are used as fenders and ice breakers. Ponton bridges should have a draw span that may be removed for the passage of drift, with booms to guide the latter into the opening.

Inspection of Bridges

All the important bridges in the area occupied by an army should be inspected at suitable intervals by specially qualified engineer-officers. This should be done at least once a month, and the results of the inspection reported to the Chief Engineer. At such inspections, any change in the nature or density of the traffic should be ascertained. It is quite as important to note the change in the condition of the bridge since the last inspection, as it is to determine its present condition.

CHAPTER XLV

CONCLUSION

THE devotion of an entire chapter specifically to the conclusion of an engineering treatise is somewhat of an innovation in technical literature, and, therefore, demands an explanation for its intrusion. The essential reason is that this is to be the author's last technical book, excepting only that after the third thousand of his "Bridge Engineering," now on sale, is exhausted, he intends to write for the second edition thereof an "Appendix" to cover the results of all bridge investigations, other than economic, which he has made (or shall have made) since that work was first issued in the summer of 1916.

The reasons why, after thirty-seven years of rather desultory book-writing, he is about to desert the field of technical literature, or, strictly speaking, that portion of it which pertains to the preparation of formal treatises, are the following:

First. In many cases the writing of technical works is an extravagant indulgence that very few engineers can afford; for not only does it necessitate the expenditure of a large amount of cold cash to prepare the manuscript of any book worthy of being considered real engineering literature, but, even after it is finished, an author sometimes has to provide all the money required for publication; the publisher deeming the many years spent and the great outlay of money used in building up the means for placing such works on the market a fair equivalent for that spent in preparing the manuscript. This is specially true in scientific books which require special means and careful study to put them properly before those they are written for.

Second. The amount of personal time that one must devote to the preparation of the MS. of any engineering work which aims to offer mainly original material, and is not, like many engineering books, a mere compilation of data gathered from standard treatises and the technical press, is simply appalling; and the diversion of such time from his professional practice is often a serious menace to the success of an engineer's business.

Third. The author feels that in presenting this treatise on bridge economics to the profession, in addition to his seven preceding books and numerous technical memoirs, he shall have almost exhausted all that he has to say in print on the subject of his chosen specialty. In other words, his message to the younger generation of engineers shall have been delivered; and he should then, with a clear conscience about having "done

his bit" towards the development of engineering science, be able to devote his remaining working years principally to serving the public in an advisory capacity on important engineering projects—provided that, before the time arrives for him to pass on, the profession shall have extricated itself from the disastrous slump into which it was plunged in 1914 by the advent of the World War.

The reader must not deduce from the preceding that the author desires to convey the impression that his labors in the field of engineering economics are over—far from it!—for he has good reason to expect that he shall be retained some day to investigate the great economic problem of "Molybdenum Steel for Bridges" in the same manner in which years ago he investigated that of "Nickel Steel for Bridges."

The economic studies of the last four years, upon which this treatise is mainly based, have been intensely interesting; but the work involved, which (somewhat unlike that of the writing of the author's *magnum opus*) has been chiefly personal, and, in spite of steady application and long working hours, exceedingly long-drawn-out, has become rather overpowering; so that, it must be confessed, the ultimate completion of the undertaking, which will not occur until the book is actually issued, will be decidedly in the nature of a relief.

In another particular the preparation of this treatise has differed from that of its predecessor, which (as a side issue, it is true,) necessitated a thorough search of engineering literature; because in this case no searching has been done, the reason being that the subject is almost entirely a new one, and, consequently, essentially independent of past records.

The author hopes that this book will prove of real service to several generations of engineers, notwithstanding the fact that, on account of changes in practice due to the rapid development of the profession, nearly all engineering books are short-lived. Such a hope is not entirely without foundation, because his little "De Pontibus," published nearly a quarter of a century ago, in spite of its contents having been absorbed by its successor, "Bridge Engineering," had a steady sale until about the end of 1920, when the publishers let it go out of print because of its low price combined with the exceedingly high cost of paper, press-work, and binding.

In conclusion, there is a suggestion which the author would like to make concerning a possible utility for this work in combination with its immediate predecessor; but he has long debated as to the advisability of offering it, fearing that some of his readers may either misconstrue his motive or charge him with undue conceit concerning the value of his technical productions. However, he has decided to run the risk; because he greatly desires that the results of the long and arduous labor which he has put upon their preparation shall be utilized to the utmost for the benefit of all future young engineers who have the ambition to specialize in bridgework. The suggestion is this:

If a young engineer of fine intelligence, great energy, unceasing persevreance, and high ambition, who has had during his technical-school course a thorough training in the theoretical portion of bridge designing and has systematically continued to read and study concerning bridgework during a practice of five or six years, devoted mainly to the various ramifications of that specialty in computation room, drafting office, manufacturing shops, and field, wishes to become a bridge expert and carry on an independent consulting practice, he can do so by adopting the following procedure: Let him devote all of his time for an entire year to the reading and re-reading of "Bridge Engineering" and "Economics of Bridgework" and to solving, by the use of these books, numerous examples of all kinds upon assumed data, so as to obtain actual experience in the computation of secondary stresses and deflections, in the designing of the various types of bridges, in the determination of economic layouts for all kinds of assumed conditions, in the expeditious making of cost estimates, in the writing of specifications for manufacture and erection of substructure, superstructure, and approaches, and in the drafting of sound engineering contracts. Such work could best be done as a post-graduate course in one of the leading technical schools; but this is not really essential, because such a young man as the one described should certainly be able to carry on his studies unaided. Again, it could be carried out well, by taking longer time, were the young man in the employ of a consulting engineer, a bridge manufacturing company, or a bridge erector. The author feels confident that, upon the completion of the training above recommended, the young man under consideration would be truly entitled to term himself a Consulting Bridge Engineer. and his success in that capacity would practically be assured.

APPENDIX

THE following is a chronologically arranged list of the author's various investigations and writings on the subject of bridge economics, extending over a period of thirty-seven years:

In 1883, a paper on "Economy in Struts and Ties."

In 1884, a chapter on "Economy" in "The Designing of Ordinary Iron Highway Bridges."

In 1891, a paper in "Indian Engineering" on "Economic Span-Lengths."

In 1897, a chapter on "True Economy in Design" in "De Pontibus."

In 1897, an elaborate investigation of the economics of cantilever bridges, published in the last-mentioned work.

In 1906, a complete investigation of the economics of steel trestles, which was later incorporated in "Bridge Engineering."

In 1907, an extensive paper on "Nickel Steel for Bridges," published by the American Society of Civil Engineers and awarded its Norman Medal.

In 1909, a paper written in French for *Le Génie Civil* and entitled "*Une Étude Économique de l'Emploi de l'Acier au Carbone à Grande Résistance pour la Construction des Ponts.*"

In 1913, a paper on "The Possibilities in Bridge Construction by the Use of High-Alloy Steels," published by the American Society of Civil Engineers, and awarded its Norman Medal.

In 1914, a paper on "Alloy Steels for Bridgework," presented to the International Engineering Congress of the San Francisco World's Fair.

In 1916, a chapter on "True Economy in Design," in "Bridge Engineering," being an expansion of a similar chapter in "De Pontibus."

In 1917, a series of three lectures on "Engineering Economics," delivered to the Engineering Alumni Association of the University of Kansas.

In 1917, a paper on "The Economics of Steel Arch Bridges," presented to the American Society of Civil Engineers, published in its "Proceedings," and awarded its Norman Medal.

In 1918, a paper entitled "Comparative Economics of Cantilever and Suspension Bridges," presented to the Western Society of Engineers and published in its "Proceedings."

In 1919, a paper on "Economic Span-Lengths for Simple Truss Bridges on Various Types of Foundation," presented to the last-mentioned society and published in its "Proceedings."

In 1919, a paper entitled "Possibilities and Economics of the Transbordeur," presented to the Institution of Civil Engineers of Great Britain, and published as the basis of two editorials in *Le Génie Civil.*

In 1920, a joint paper with Mr. H. Malcolm Priest on "Comparative Economics of Continuous and Non-Continuous Trusses," presented to the Engineers' Society of Western Pennsylvania and published in its "Proceedings."

In 1920, a paper for the same society on "Comparative Economics of Wire Cables and High-Alloy-Steel Eyebar-Cables for Long-Span Suspension-Bridges," and published in its "Proceedings."

In 1920, a memoir entitled "*De l'Emploi Économique des Alliages d'Acier dans la Construction des Ponts,*" published in condensed form by the *Académie des Sciences* on July 12, 1920, and in full by *Le Génie Civil* on July 24, 1920.

In 1920, a paper entitled "Bridge *versus* Tunnel for the Proposed Hudson River Crossing at New York City," presented to the American Society of Civil Engineers and published in its "Proceedings."

In 1920, an as-yet-unpublished paper on "Economics of Reinforced-Concrete, Steam-Railway Bridges."

In 1920, a paper on "Economics of Movable Spans," presented to the American Railway Engineering Association, but not yet published.

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